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Secondary stresses in trusses with rigid joints, special application to glued wooden trusses

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SECONDARY STRESSES IN TRUSSES WITH RIGID JOINTS, SPECIAL APPLICATION TO GLUED WOODEN TRUSSES

by

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INTRODUCTION

To a farmer, buildings represent working tools which provide conditions favorable to the increase in income from his business enterprise. In order to do this effectively, the buildings should not only provide satisfactory functional requirements and contribute to efficiency in operation, but they should also be constructed substantially to minimize losses from wind and fire.

Wood continues to be the principal construction material because it is relatively low in cost, is readily available, and can be fabricated on the farm. Poor joints and improper design, however, often limit the structural effectiveness of farm buildings. The structural application of glue gives promise of a relatively inexpensive fastening with strength at the joint comparable to other portions of the structure. The use of fixed glued joints could result in secondary stresses different from stresses calculated by simple mechanics.

This investigation deals primarily with the presence and magnitude of these "secondary stresses" and their influence upon the design of timber trusses adapted to farm construction. Other considerations incidental to the analysis of secondary stresses relate to problems in loading full-scale trusses and measuring strains in wood. An analysis of these problems is included as contributory to the general objective.

Objectives

1. To find methods of measuring strains in members of timber trusses and to select or construct instruments for reading and recording these strains.
2. To study the distribution and magnitude of secondary stresses developed in glued timber trusses and practical methods for making allowances for them in design. Also to compare measured stress with theoretical computed stress.
3. To determine properties of plywood most desirable for this type of construction and its ability to resist combined direct and twisting stresses.
4. To select acceptable values for wind pressures and other live loads on structures being considered in this thesis.
5. To study loading methods for quickly and accurately applying loads to full-scale trusses.

Review of Literature

The Review of Literature was made in six parts:

1. Secondary stresses
2. Design loads on roofs
3. Use of glue in construction
4. Testing wooden trusses
5. Design conditions
6. Determining stresses

Secondary stresses

Von Abo (52) indicates that the calculation of secondary stresses would lead to much better distribution of metal in bridges, and would result in a structure of more uniform strength. He feels that part of the factor of safety, or as he refers to it, the "factor of ignorance", was due to secondary stresses. Apparently, designers have not considered these stresses as they are not sure how or where they act.

According to Von Abo, secondary stresses are those in the fiber of the material, different from those calculated by dividing the axial stress by the cross-sectional area of the member. In the present investigation bending caused by inter-panel point loading is also considered as a cause of primary stress. These stresses vary from the gravity axis of the material to the outer surface. Von Abo classifies secondary stresses into the following four types:

1. Secondary stresses arising from rigid or partly rigid connections.
2. Secondary stresses arising from the beam action of members.
3. Secondary stresses developed in axially loaded members.
4. Secondary stresses due to faulty design.

The author indicates that the primary stress usually considered by designers is the average of the actual stresses found at the four corners of each member. The secondary stress is the difference between this average and the actual stress that is found in the member concerned. Most designers disregard secondary stresses, assuming they are not critical and that a factor of safety can be depended on to provide adequate strength.

The author then outlines several methods of analysis proposed by other engineers for the determination of secondary stresses.

Fuller and Kerekes (18, p. 530) determine the joint displacements by a Williot diagram; then by the slope-deflection equation, they compute the moments caused by the deflection. These moments are distributed by the moment distribution process to determine the magnitude of the moment in each member. A shorter method is also proposed:

> A second method, "carry-over-distribution", by-passed some of the steps on the assumption that all unbalanced moments can be distributed at the end of the process, because each succeeding unbalanced moment distributes in the same ratio among the members at any one joint. (18).

A distribution-carry-over factor equal to the distribution factor multiplied by the carry-over factor is applied to the unbalanced moment at each joint.

Design loads on roofs

The information from various investigators concerning the design factors for wind loads on low roofs is plotted as shown in Figure 1. The results of Irminger and Nokkentved's attempt to determine the effect of roof slope on the wind pressure coefficients on roofs are shown in Figure 2.

α	h/l	SYMBOL	INVESTIGATOR
45°	r = .50	I & N	Irminger and Nokkentved
40°	r = .41	E	Eiffel
20°	r = .18	D & H	Dryden and Hill (Bowstring)
33.7°	r = .33	S	Smith
45°	r = .50	U	Bounkin and Tcheremoukhin
40°	r = .42	F	Flachbart
		F & O	Fenten and Otis
45°	r = .50	X	Chein, Feng, Wang, Siao

$q = .002558\, V^2$ $r = h/l$

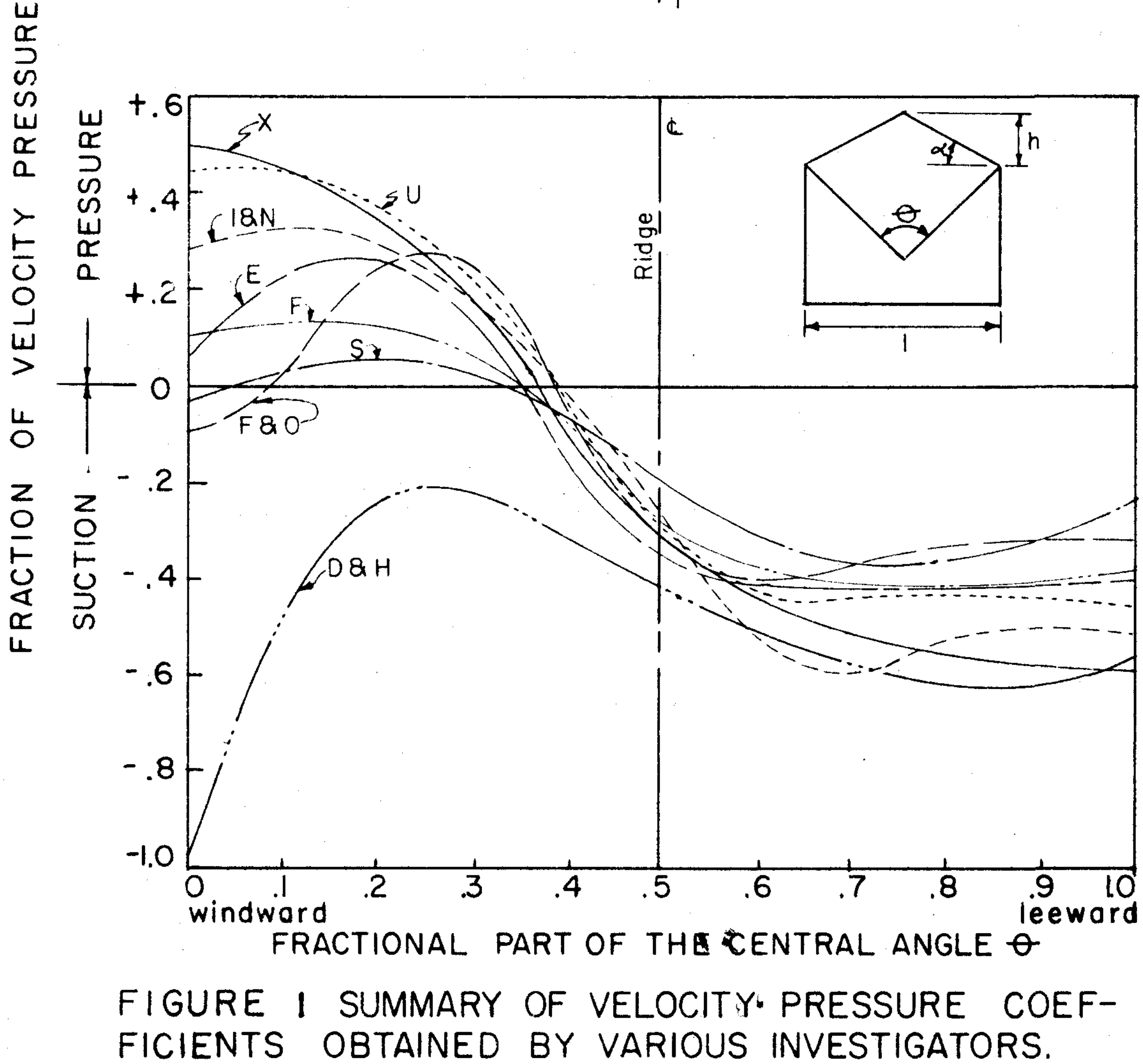


FIGURE 1 SUMMARY OF VELOCITY PRESSURE COEFFICIENTS OBTAINED BY VARIOUS INVESTIGATORS.

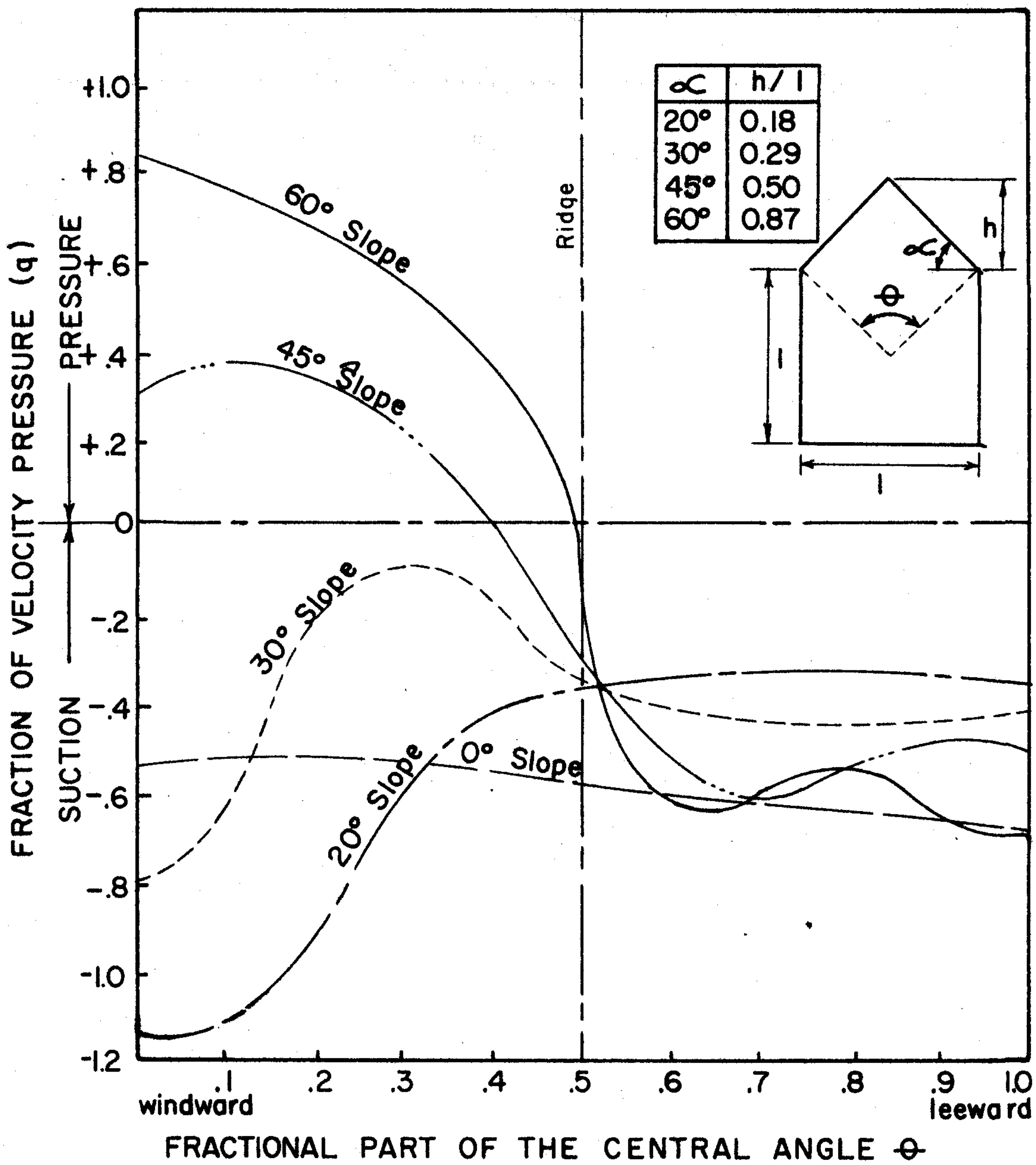


FIGURE 2 THE EFFECT OF ROOF SLOPES ON VELOCITY PRESSURE COEFFICIENT AS DETERMINED BY IRMINGER AND NOKKENTVED.

Figures 1 and 2 indicate a wide variation in the pressure coefficients although the general pressure pattern is the same -- a negative pressure or suction on the leeward side. From this information, the following coefficients were determined for use with a wind pressure in psf:

Wind:	windward side wall coefficient	+0.8
	lower 1/2 of windward roof	+0.3
	upper 1/2 of windward roof	-0.2
	upper 1/2 of leeward roof	-0.5
	lower 1/2 of leeward roof	-0.5
	leeward wall	-0.4

Barre and Sammet (3) recommend a minimum allowance of 20 psf for snow load. However, in northern areas of the United States, this allowance should be increased according to figures prepared by the National Bureau of Standards (32). For Iowa and most other states in the North Central Region, the design snow load would be 25 psf.

The use of glue and wood in construction

For most farm construction wood is commonly used, although in constructing trusses a farmer is usually unable to design joints strong enough to develop the full strength of the wood.

Glesinger (22, p. 12) states:

> Wood will become the characteristic raw material of our civilization. Because it has three attributes which make it unique among raw materials - 1. Wood is universal. 2. Wood is abundant. 3. Wood is inexhaustible. Indications were that the waste in

> use of wood is so large that if all this waste were processed, each person in the United States could be furnished with four tons of wood.

Tiemann (47, p. 237) states:

> Weight for weight, dry wood in the form of beams is stronger and stiffer than steel -- if the full tensile strength of wood could be brought into play, it would be very much stronger than the same weight of steel.

Markwardt (29, p. 442) states:

> Perhaps no single factor has been more responsible for the significant advance in the field of modern wood construction than the development in glue and gluing techniques.

Markwardt also indicates that the marked increase in the use of wood as a structural material is attributed to eleven developments, the first three of which are:

1. Improvement in joints and fastenings, usually the critical features of structural timber design.
2. Better structural grades affording more precise strength evaluation or more uniform quality.
3. Improvement in glue and gluing techniques, including the development of synthetic resins that afford essentially water-proof joints.

Considerable information is available on the various types of glue. Hamlin (24) and Palmer (36) have compiled a complete review of literature on this subject. The structural glues most commonly used for farm construction are casein and resorcinol resin. Casein is used in joints somewhat protected from moisture, as it is not waterproof. Resorcinol resin glues have been developed since the war and are waterproof. They can also be used in joints where their gap-filling qualities are needed for a thick glue line.

In experiments conducted by Dosker and Knauss (10) to determine the possibility of laminating white oak for use in boat timbers, the exposure to which these members were subjected made it imperative that the glue be waterproof.

Dosker and Knauss also determined the minimum bending radius for various thicknesses of Douglas fir and white oak. When they used phenol, melamine and resorcinol low-temperature glues they obtained highest strengths with a moisture content between 8% and 15%. The members were glued at the same moisture content anticipated under loading conditions.

De Lollis, Rucker and Wier (9) conclude that resorcinol resin and casein gave strength values ranging from 590 to 1940 psi with hard rubber, paper-phenolic laminate, and birch.

When Martin (30) tested 10'-8" lengths of solid beams, seven-ply commercial rafters, and straight laminated glued beams of six and seven plies, he found that the horizontal shearing strength of cold water casein glue exceeds that of wood. He concluded that after eight years of storage conditions, the quality of the glue had not been seriously affected. He found shear strength values up to 550 psi, giving a factor of safety of approximately 4 when used with yellow pine.

Skinner (41) found that joints made with resorcinol-resin glue and Douglas fir lumber at 16 per cent moisture content were approximately as strong as similar joints made from wood at 7 per cent moisture content. Joints made with the wood at a moisture content of 30 per cent and tested at 22 per cent were approximately 70 per cent as strong as those made from the drier wood. These tests showed also that glue joints may be made successfully under field conditions over a wide range in moisture content.

Skinner further concluded that a nail spacing of 4 inches center to center with 8d nails was sufficient and that design loads would be limited only by the allowable fiber stress of the joint members for nominal two-inch Douglas fir members and nominal one-inch white pine gusset plates.

Palmer (36) determined the effect of the size of the member on the strength of the joint, the effect of dimensional changes due to moisture change, and the effect of warping on strength. As his test samples, he used joints made with 1" x 4", 1" x 6", 1" x 8", 2" x 4", 2" x 6", and 2" x 8" material; he made the joints by placing the grain of the two pieces perpendicular to each other. He concludes: (36, p. 80)

> (1) Glued joints made from members with a width of 4 inches or less at 12 per cent moisture content showed no appreciable loss in strength when tested at 9 per cent moisture content. (2) Glued joints made from members of 6 inch and 8 inch width at 12 per cent moisture content showed a marked decrease in strength when tested at 9 per cent moisture content. (3) A cycle of change in moisture content has no effect on the strength of glued joints regardless of their size so long as they

> are tested at the same moisture content. (4) Applying a factor of safety of 2 to the average strength of the weakest group in each size bracket, the following design stresses are recommended for cross-grained glued joints to be located in unheated farm buildings
>
> | | |
> |---|---|
> | 4" x 4" or less | 100 psi |
> | 6" x 6" | 75 |
> | 8" x 8" | 50 |

The use of photo-elastic models to determine stress patterns was investigated by Hamlin (24, p. 114), who arrived at the following conclusions:

> (1) Allowing a safety factor of at least 2, a length of 5 inches for any width of glued joint from one inch to 8 inches will permit design of the joints on a basis of an allowable compressive stress of 1700 psi for Douglas fir. The stress in the glue line may be neglected when using the above allowable working stress for joints 5 inches in length or greater.
>
> (2) An increase in length of joint beyond 5 inches serves only to increase the safety factor of a glued joint since the allowable compressive working stress of Douglas fir does not permit design loads which would result in high shearing stresses in the glue line.
>
> (3) For glued joints, less than 5 inches in length, an allowable glue line shearing stress of 200 psi will provide a safety factor of 2 or above.
>
> (4) The load applied to glued joints is not distributed uniformly over the glued area.
>
> (5) Shear stresses in a glued joint are highest at points closest to the application of load.

When the test joints were loaded, Hamlin restrained the double members from moving laterally. The effect of the restraint was a higher unit stress at the glue line. Since the line of force is as shown in Figure 3 the horizontal

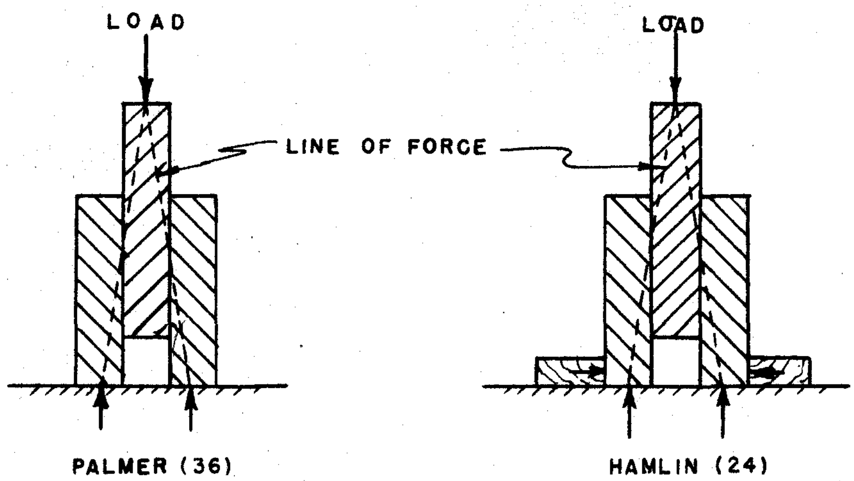


FIGURE 3 SCHEMATIC DIAGRAM OF TEST SPECIMEN USED BY HAMLIN AND PALMER.

component of the force is taken by the horizontal support. The fact that Palmer did not support his test joints against lateral movement may account for lower values of unit stress than reported by Hamlin.

Giese and Henderson (21) recommend, as a practical shear design stress in farm structures, 430 psi for nominal two-inch Douglas fir main members with nominal one-inch yellow pine or white pine gusset plates when fabricated with casein glue. These investigators also recommend a torsional stress design value of 215 psi for parallel-grained joints.

De Bruyne (8), reporting on stress concentrations in glued metal and using a joint similar to the joints used by Hamlin, verified his theoretical analysis with experimental data.

Martin (30) tested laminated bent rafters that had been constructed some years before and exposed to service conditions. He found that glue on wood developed adequate shear strength to provide a factor of safety somewhere near 5.

The U. S. Forest Service (51, p. 105) makes the following statement:

> Satisfactory adhesion of glue to wood is obtained at any moisture content of the wood up to 15 per cent and even higher with water resistant glues. Large changes in the moisture content after gluing however, develop stresses that may seriously weaken both the wood and the joint.

It also conducted tests using nails for applying pressure to determine the value of shear for glue. With one 8 d nail per 8 square inches of glue area, values as high as 1015 psi were recorded. However, a value of 120 psi was recommended, allowing a factor of safety of 2.

Hamlin (24), after testing a large number of double lap joints for shear to determine the effect of size and shape on the shear strength, recommended an allowable glue line shearing stress of 200 psi, which permitted a safety factor of 2 for joints more than 5 inches long.

From his study of moisture effects on parallel-grained joints, Fry (17) concluded that the effects of variation in wood structure, gluing pressure, and other uncontrollables largely outweigh the effects of moisture changes.

Testing timber trusses

There is a very noticeable shortage of literature concerning the testing of timber trusses. Some work has been done with steel.

Gloss (23) tested a new truss design which seemed to be more economical and require less material than the old Pratt truss. The Gloss-Lank truss had a horizontal top chord over approximately the center one-third and sloping top chords extending from this horizontal chord to the ends of the lower chord. The truss was supported on a concrete column at one end and on a wooden column on the other. The load was applied by means of a large sand box suspended from stirrups placed over the top of the truss at the panel points. The truss was loaded up to four times the design loads.

In order to test the deflection, a fine piano wire, run over a pulley, was stretched under the truss and kept at a constant tension by a weight attached to the wire.

Determination of stress in members

Some method of measuring stress in wood is necessary in order to analyze the stresses in the various members.

Of the strain gages now available, the simplest is a scratch gage, which consists of two reference points secured to the member being tested and a pointer which scratches the surface of the template as the member is strained. This type of gage is suitable for use on wood and, due to its simplicity, rather inexpensive. Reading the strain at various loadings is very difficult, however, as the template must be magnified in order to measure the distance moved.

The Huggenburger mechanical strain gage is commonly used for measuring stress in metal parts and is reasonably accurate. It has two points which must be firmly embedded in the member; however, it is difficult to choose a section of a wooden member which is representative of the piece. Furthermore, the time involved in reading the strains at approximately one hundred points is too long. The Porter-Lipp gage is similar to the Huggenburger gage.

Lee (28) described the use of SR-4 electric resistance strain gages for measuring strain in metallic materials. This gage records the change in resistance of a specially constructed wire as it is stretched or compressed. One big advantage of this method of strain measurement is that electrical wire leads can be connected to the gage and run some distance to a central point for recording. This arrangement is convenient where several members are under consideration and where strain measurements at various conditions of loading are required.

Lee's relationship for stress shows that stress is a function of the modulus of elasticity, the strain both parallel and perpendicular to the axis of the member, and to the Poisson ratio in the two directions. However, if the Poisson ratio μ_{vu} is small and the strain e_v at right angles to the member is small, the unit stress is equal to the modulus of elasticity multiplied by the unit strain.

Ernst (13) listed and illustrated applications of the SR-4 to testing stresses in wood. The majority of the SR-4 applications in wood testing were done with the SR-4 scanning recorder to obtain sufficient points for an adequate stress pattern. When determining buckling loads and using ten or fewer gages, the control box or portable strain indicator, combined with a manual switching unit, was found more convenient.

Although no special difficulties were encountered in the use of such gages on wood, Ernst warned that care must be taken to choose the proper strain magnification for the work.

Hetenyi (25), in his discussion of SR-4 gages, reports very little creep effect in the gages under continual strain where Duco cement is allowed to dry at least one day. He was concerned, however, with the use of SR-4 gages only on metal surfaces.

THE INVESTIGATION

Preliminary Considerations

Measuring strain in wood

The Review of Literature indicated that very few references to the measurement of internal streses in wood were available. Using select grade lumber, Stern (43) found that the stress distribution over the section of the member was essentially proportional to the strain. In some cases, the neutral axis shifted to the tension side of the beam, especially if the growth rings lay in the direction of the applied load. An inclination of the growth rings or irregularities in the growth ring may prevent a shift of the neutral axis.

In his tests, Stern used dial gages to measure strain at various points over the beam section. These gages were satisfactory for laboratory use where strain at points was required, but they could not be used for truss testing, as the panel points move and thus upset the readings.

In the present investigation preliminary laboratory tests were run on a one and three-fourth inch square beam fifteen inches long, carrying a concentrated load at the center. The load was gradually increased and the strain recorded.

At two different loads, with a modulus of elasticity of 1,330,000 psi (a value determined for a specimen of wood by SR-4 strain gages), the strain was measured and the stress calculated as follows:

Table 1

Data from First Test to Determine the Accuracy of SR-4 Strain Gages

	Test I	Test II
Load	300 lb	850 lb
Calculated stress	1315 psi	3720 psi
Measured stress	1330 psi	3630 psi
Difference	1.14%	2.42%

In a second series of tests, one and three-fourths inch square beams 18 inches long were loaded with a concentrated load in the middle of the span. An Ames dial gage was used to measure the vertical deflection at midspan. Each beam was loaded both parallel to the growth rings and perpendicular to the growth rings. The strain at the bottom of the beam and the magnitude of the load were recorded. Stresses were calculated by two methods,

(1) $S = Ee$ (2) $S = \frac{Mx6}{bd^2}$

where S = Unit stress psi

e = Unit strain inches per inch - measured by SR-4 strain gage

$M = \frac{PL}{4}$ (P in pounds, L in inches)

b = Width of the beam in inches

d = Depth of the beam in inches

E = Modulus of elasticity of 2,030,000 psi calculated from a compression specimen using SR-4 gages. See Figure 4.

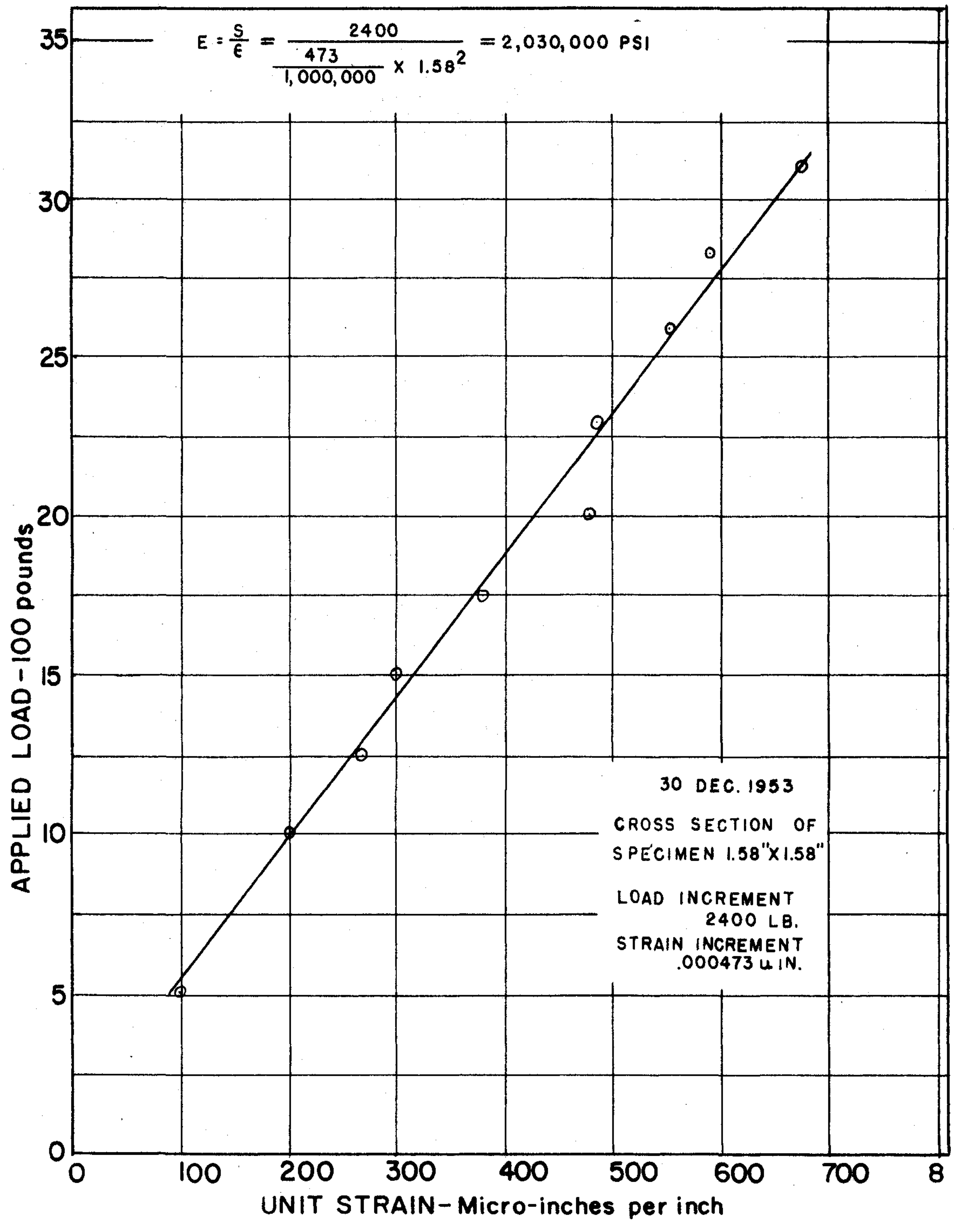


FIGURE 4 MODULUS OF ELASTICITY DETERMINATION FOR DOUGLAS FIR IN COMPRESSION.

The following table summarizes the results of the tests:

Table 2

Results of Tests on Douglas Fir Beams to Check the Accuracy of SR-4 Strain Gages

	Stress by SR-4 gage psi	Stress by calculating psi	Difference % of calculated
Beam I			
Load normal to rings	4530	4600	1.5
Load tangential to rings	5000	4680	6.8
Beam II			
Load normal to rings	4850	4580	5.7
Load tangential to rings	4880	4580	6.5

Another check was made to determine the accuracy of SR-4 gages. This involved calculating the modulus of elasticity by two methods, (1) measured deflection and (2) measured strain.

Table 3

Summary of Tests to Determine Modulus of Elasticity Based on Measured Deflections versus Measured Strain

	Strain by SR-4 in/in	Defl. in.	"E" (bending)	
			By SR-4*	By Defl.**
Beam I				
Load normal to rings	2230	.092	2,060,000	1,690,000
Load tangential to rings	2460	.092	1,910,000	1,750,000
Beam II				
Load normal to rings	2400	.092	1,920,000	1,700,000
Load tangential to rings	2400	.092	1,910,000	1,700,000
Average			1,950,000	1,710,000

$^{*}E = \frac{S}{e}$ $^{**}E = \frac{wl^3}{48\Delta I}$

From these tests, it seemed logical to assume that the SR-4 gage could be used to measure strain in wooden members. The direction of stress must be known, however, to be certain that maximum values are read. The direction could be determined in members with direct tension, compression, or bending, but on the gusset plates, this direction could not be determined quickly.

Lee (28) presented the mathematical solution for determining principal stress when the strain in three directions is known. His relationship is as follows:

$$\sigma_{1,2} = \frac{E}{2}\left[\frac{e_a - e_b}{1+\mu} \pm \frac{1}{1+\mu}\sqrt{2(e_a - e_b)^2 + 2(e_b - e_c)^2}\right]$$

where $\sigma_{1,2}$ = principal stresses, $e_{a,b,c}$ = measured strains along three known axes a, b, c, and μ = Poisson ratio.

Determination of modulus of elasticity for wood

In a material whose modulus of elasticity "E" is the same in all directions, Lee's formula can be used. In preliminary tests on samples of Douglas fir and plywood, it was found by the present investigator that the modulus of elasticity varied with the direction of the grain in both the plywood and the Douglas fir, as shown in Table 4.

The Forest Products Laboratory (51), after having run a series of tests to determine the "E" value for wood, indicated the same design value both parallel and perpendicular to the grain for No. 1 grade Douglas fir.

Table 4

Modulus of Elasticity (Compression) for Plywood* and Douglas Fir Determined with SR-4 Strain Gages and the Comparative Values as Determined by the Forest Products Laboratory

Material and load	Sample I x 1000	Sample II x 1000	Ave. x 1000	F. P. L. x 1000
D-Fir load parallel to grain	1800	2060	1930	2280
D-Fir load perpend. to grain	478	600	539	113
Plywood load parallel to grain in outer lamination	975	975	975	
Plywood load perpend. to grain in outer lamination	633	594	610	

*Exterior grade A-A plywood

According to the preceding tests on sample taken from the truss, the modulus of elasticity to be used in the following calculations and investigation is the average.

$$\frac{1{,}930{,}000 + 1{,}950{,}000}{2} = 1{,}940{,}000 \text{ psi}$$

Theoretical Analysis

Type of truss to be tested

The truss to be investigated is a standard type used by the Timber Engineering Company as shown in Figure 5. A check of the plans commonly used in the Midwest revealed considerable variation in the truss spacings. Since one of the prime interests to most farmers is economy, cost estimates were made of the various trusses based on the following assumptions:

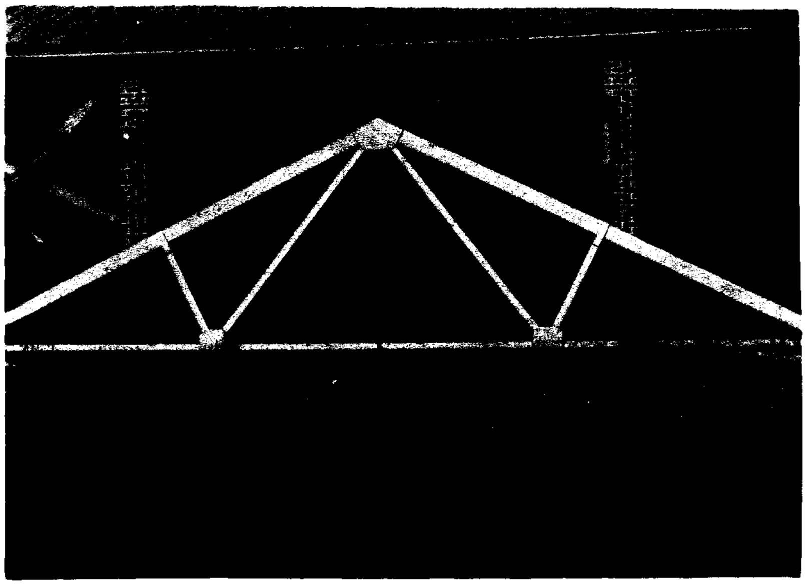


Figure 5. The Fink truss used in the investigation

1. That solid roof sheathing is used,
2. That prices quoted are FOB Ames, Iowa,
3. That designs are taken from plans proposed for the Midwest region,
4. That cost figures are based on a square foot of floor space covered by the roof.

The results appear in Table 5.

Table 5

Cost per Square Foot Covered by Truss Types and Sizes

Truss spacing	2'o.c.	4'o.c.	6'o.c.	8'o.c.	12'o.c.	16'o.c.
Teco						
30' span	$.29			$.18		$.27
36' span	$.46				$.28	
MWPS						
24' span	$.19				$.15	
36' span	$.22		$.17		$.20	
Thesis Plan						
36' span		$.15*				

*Calculated on the basis of using timber connectors

From this information, it appears that either a 4-foot or 6-foot spacing for the rafter trusses is most economical. Ceiling joists must be used between the wide-spaced trusses, while with narrower spacings the lower chords serve as ceiling joists. The additional joists increase costs. Since the 6-foot spacing is also inconvenient if standard sheet materials, are used, so the 4-foot spacing seems to be the most practical.

A 36-foot span was used, as it is the longest span commonly called for in farm buildings although there are a small number of plans that show longer spans.

A 36-foot span can be used for one-story dairy barns, machine sheds, cattle sheds, or other wide structures.

Loading arrangement and roof design load

The load on a farm building roof is uniformly distributed. However, in this investigation it was planned to load the top chords at the panel points and at the third points of the top chords. The relative moments caused by these two loading conditions are as follows:

For a beam with a load P at the one-third points the fixed end moment is equal to 2/9 PL

however, since $P = W/3$

therefore, $M = 2/27\ WL$

For a continuous beam loaded uniformly, the fixed end moment is equal to

$$1/12 \times WL = 2/24\ WL$$

Consequently it can be seen that the bending moment at supports caused by two concentrated loads at the one-third points is eleven per cent less than that caused by a uniformly distributed load.

The truss used in this investigation was designed with a third-point loading on the top members.

Computing axial stresses

The following design loads were used in the present design:

Dead load --- 5.0 PSF of roof surface

Snow load --- 25.0 PSF of horizontal projected surface

Wind load --- 25.0 PSF of roof surface

(70 MPH wind and gust factor of 0.4)

The pressure coefficients as shown in the Review of Literature were applied to the basic wind load to give loads on the roof surfaces.

With these unit loads, the stress in each member was calculated for two conditions of loading, namely, dead plus wind and dead plus snow (Figure 6). It was found that the maximum compression and tension stresses both occurred under the dead plus snow loading. Thus, the design was made with dead plus snow loading. The wind loading reversed the direction of stress in some members. However, the magnitudes of these stresses are small.

As the results of this investigation are intended for use on a farm or in a small plant, the minimum size member is a nominal 2" x 4" section. With a 2" x 4" minimum section, the unit axial stress in the members was as follows:

L_0U_1 = -304 psi	U_1L_1 = -170 psi	U_3L_2 = -170 psi
U_1U_2 = -266 psi	L_0L_1 = +565 psi	L_2L_3 = +565 psi
U_2U_3 = -266 psi	L_1L_2 = +374 psi	L_1U_2 = +187 psi
U_3L_3 = -304 psi	U_2L_2 = +187 psi	

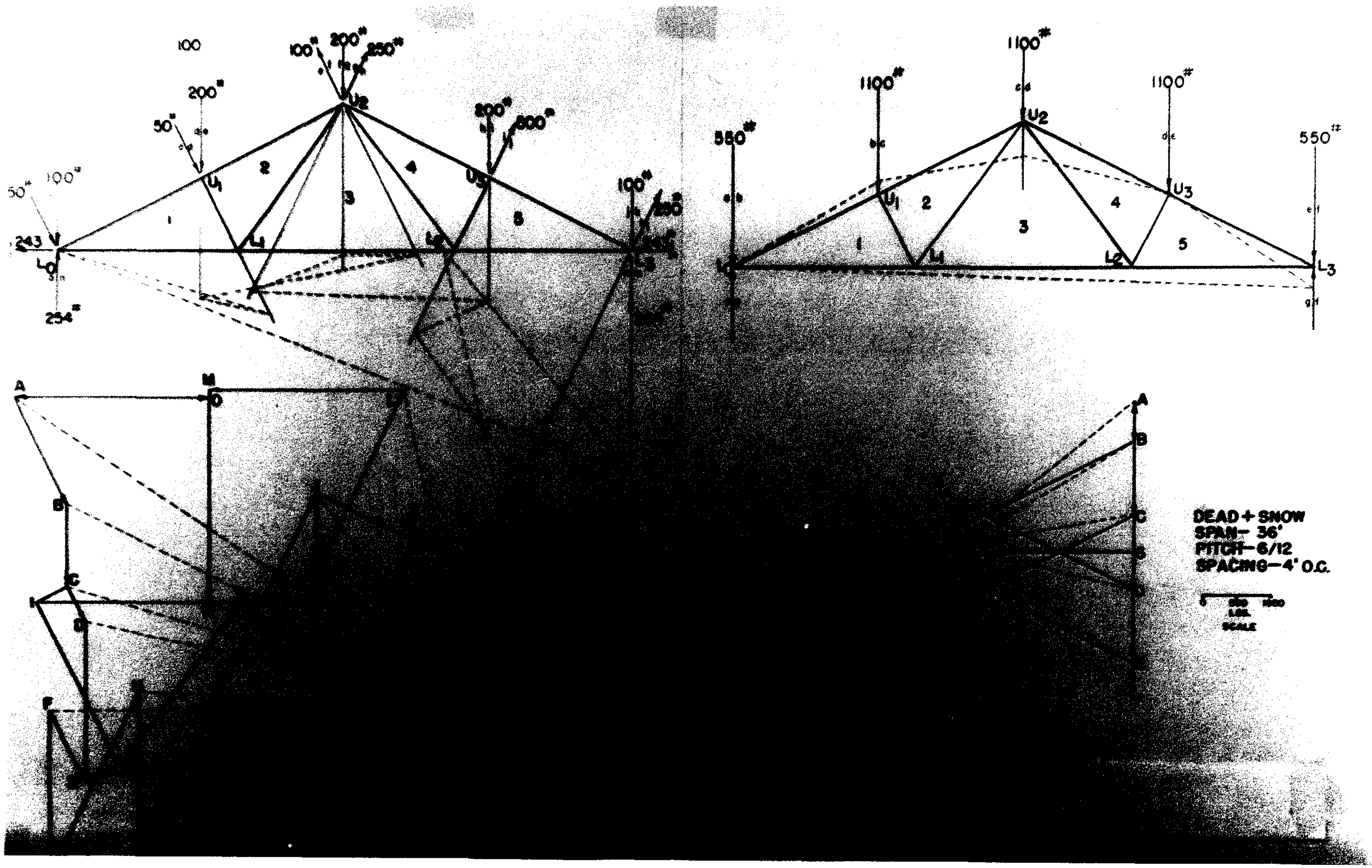


Figure 6. Graphic solution for stresses in members due to loads as shown

Computing moments due to continuous beam action in top chords

In addition to the moments caused by deflection, there are those caused by the inter-panel point loading. The formula for the fixed end moments on these members is equal to

$$M = 2/9\ PL = 2/9 \left(\frac{18}{20.2} \times \frac{1100}{3}\right) \times (10 \times 12) = 8700 \text{ in.lb.}$$

where P = Load applied at one-third points of the upper chords of the truss in pounds

and L = Length between panel points in inches

Moment distribution

These moments were distributed throughout the truss members by the principle of moment distribution. The results are shown in Table 6.

Checking deflection of roof sheathing

In the design of farm buildings two methods of framing can be used in conjunction with the roof truss under construction. First, when shingles are used for roofing, one-inch sheathing is nailed directly to the upper chords of the truss. Second, when metal roofing is desired, nominal 2" x 4" nailing girts spaced 2 feet apart, center to center, are nailed to the truss chords. The metal roofing is nailed to the girts.

With the trusses spaced four feet center to center the deflection Δy of two types of the roof decks was calculated to determine whether or not it would be excessive for the respective types of roof coverings.

Table 6

Moment Distribution for Moments Caused by Bending in Top Chords

	L_0		U_1			L_1			
	L_0U_1	L_0L_1	U_1L_0	U_1L_1	U_1U_2	L_1L_0	L_1U_1	L_1U_2	L_1L_2
D.F.	91	9	45	10	45	20	44	20	16
FEM	-8700	00	+ 8700	00	- 8700	00	00	00	00
Bal (1)	-7920	-780	00	00	00	00	00	00	00
C.O.	00	00	- 3960	00	00	-390	00	00	00
Bal (2)	00	00	+ 1782	+396	+ 1782	+ 78	+172	+78	+62
C.O.	+ 991	+ 39	00	+ 86	00	00	+198	00	-31
Bal (3)	- 938	- 92	- 39	- 9	- 38	- 33	- 74	-33	-27
C.O.	- 19	- 16	- 469	- 37	00	- 46	- 4	00	+14
Bal (4)	+ 31	+ 4	+ 228	+ 50	+ 228	+ 7	+ 16	+ 7	+ 6
C.O.	+ 114	+ 3	+ 15	+ 8	00	+ 2	+ 25	00	- 3
Bal (5)	- 107	- 10	- 11	- 2	- 10	- 5	- 10	- 5	- 4
C.O.	- 5	- 2	- 53	- 5	00	- 5	- 1	00	+ 2
Bal (6)	+ 6	+ 1	+ 26	+ 6	+ 26	00	+ 4	00	00
C.O.	+ 13	00	+ 3	+ 2	00	00	+ 3	00	00
Bal (7)	- 11	- 2	- 2	- 1	- 2	00	- 3	00	00
	- 857	+857	+11180	-494	-10686	+392	-326	-47	-19

Table 6 Continued

	U_2				L_2				U_3			L_3	
	U_2U_1	U_2L_1	U_2L_2	U_2U_3	L_2L_1	L_2U_2	L_2U_3	L_2L_3	U_3U_2	U_3L_2	U_3L_3	L_3U_3	L_3L_2
D.F.	45.5	4.5	4.5	45.5	16	20	44	20	45	10	45	91	9
FEM	+8700	00	00	-8700	00	00	00	00	+ 8700	00	- 8700	+8700	00
Bal (1)	00	00	00	00	00	00	00	00	00	00	00	+7920	+780
C.O.	00	00	00	00	00	00	00	+390	00	00	+ 3960	00	00
Bal (2)	00	00	00	00	- 62	- 78	-172	- 78	- 1782	-396	- 1782	00	00
C.O.	+ 991	+ 39	- 39	- 992	+ 31	00	-198	00	00	- 86	00	- 992	- 39
Bal (3)	00	00	00	00	+ 27	+ 33	+ 74	+ 33	+ 33	+ 9	+ 39	+ 938	+ 93
C.O.	- 19	- 16	+ 16	+ 19	- 14	00	+ 4	+ 46	00	+ 37	+ 469	+ 19	+ 16
Bal (4)	00	00	00	00	- 6	- 7	- 16	- 7	- 228	- 50	- 228	- 32	- 3
C.O.	+ 114	+ 3	- 3	- 114	+ 3	00	- 25	- 2	00	- 8	- 16	- 114	- 3
Bal (5)	00	00	00	00	+ 4	+ 5	+ 10	+ 5	+ 11	+ 2	+ 11	+ 107	+ 10
C.O.	- 5	- 2	+ 2	+ 5	+ 2	00	+ 1	+ 5	00	+ 5	+ 53	+ 5	+ 2
Bal (6)	00	00	00	00	00	00	- 4	00	- 26	- 6	- 26	- 6	- 1
C.O.	+ 13	00	00	- 13	00	00	- 3	00	00	- 2	- 3	- 13	00
Bal (7)	00	00	00	00	00	00	+ 3	00	+ 2	+ 1	+ 2	+ 11	+ 2
	+7606	- 24	+ 24	-7606	+ 19	+ 47	+326	-392	+10686	+494	-11180	+ 857	-857

For solid one-inch sheathing:

$$I = \frac{11\ 1/2 \times (7/8)^3}{12} = .642 \text{ in.}^4$$

$$\Delta y = \frac{5wl^4}{384EI} = \frac{5 \times 25 \times 4 \times 48}{384 \times 1,500,000 \times .642} = .149 \text{ in.}$$

For 2" x 4" nailing girts 24 inches on center:

$$I = \frac{3\ 6/8 \times (1\ 5/8)^3}{12} = 1.30 \text{ in.}^4$$

$$\Delta y = \frac{5 \times 25 \times 2 \times 4 \times 48^3}{384 \times 1,500,000 \times 1.30} = .150 \text{ in.}$$

From these values, it was found that the maximum deflection under full load is small. For this reason, either type of roof decking can be used without excessive sagging between the rafters.

Glue areas for joints

In the Review of Literature, DeLollis (9) quoted results in which the shear values for glue varied up to 1560 psi with an average of 530 psi. Hamlin (24) recommended a value of 200 psi. Palmer (36) suggested a value of 100 psi, which allowed a factor of safety of two over his weakest joint.

From these results, it seems that a value of 200 psi is practical for farm construction. The allowable horizontal shear in Douglas fir recommended by the Forest Products Laboratory is 95 psi, which would be the limiting value in the joints. Using this value for design would allow a safety factor of two for the glue. Table 7 summarizes the glue contact area required for the various members based on the value of 95 psi.

Table 7

Minimum Glue Contact Area* Required between the Gusset Plates and the Individual Members

Member	Area - in^2	Member	Area - in^2
L_0U_1	39.0	L_2L_3	34.9
U_1U_2	34.1	U_1L_1	10.5
U_2U_3	34.1	U_2L_1	11.6
U_3L_3	39.0	U_2L_2	11.6
L_0L_1	34.9	U_3L_2	10.5
L_1L_2	23.2		

*These figures do not include the effect of twisting on the members.

Calculating change in length in members

The change in length of each member due to the axial stress was calculated from the relationship:

$$\lambda = \frac{SL}{AE}$$

$\lambda =$ change in length in inches

$S =$ axial stress in pounds

$L =$ length of member in inches

$A =$ area in square inches

$E =$ 1,940,000 psi (from test results)

Table 8 shows a tabulation of the above factors and the resulting change in length.

Table 8

Data Used for Calculating the Change in Length of Truss Members

Member	A in.^{2}	L in.	S lbs.	$\frac{SL}{AE}$ in.
L_0U_1	12.19	120.0	-3700	-.0188
U_1U_2	12.19	120.0	-3240	-.0165
U_2U_3	12.19	120.0	-3240	-.0165
U_3L_3	12.19	120.0	-3700	-.0188
L_0L_1	5.89	134.5	+3320	+.0391
L_1L_2	5.89	165.0	+2200	+.0318
L_2L_3	5.89	134.5	+3320	+.0391
U_1L_1	5.89	60.0	-1000	-.0053
L_1U_2	5.89	134.5	+1100	+.0130
U_2L_2	5.89	134.5	+1100	+.0130
L_2U_3	5.89	60.0	-1000	-.0053

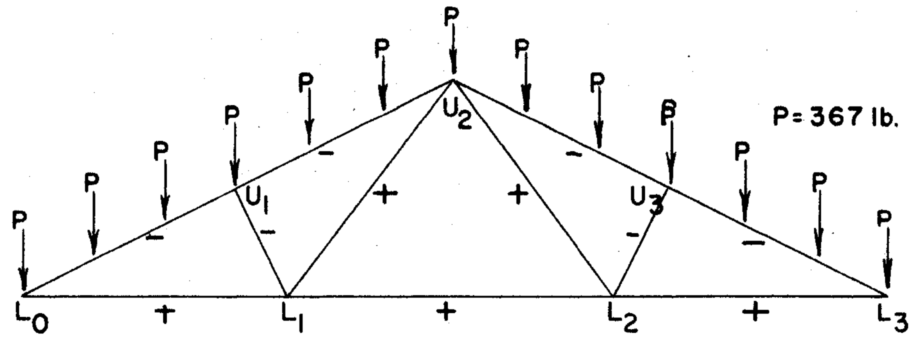
Calculating panel point deflection

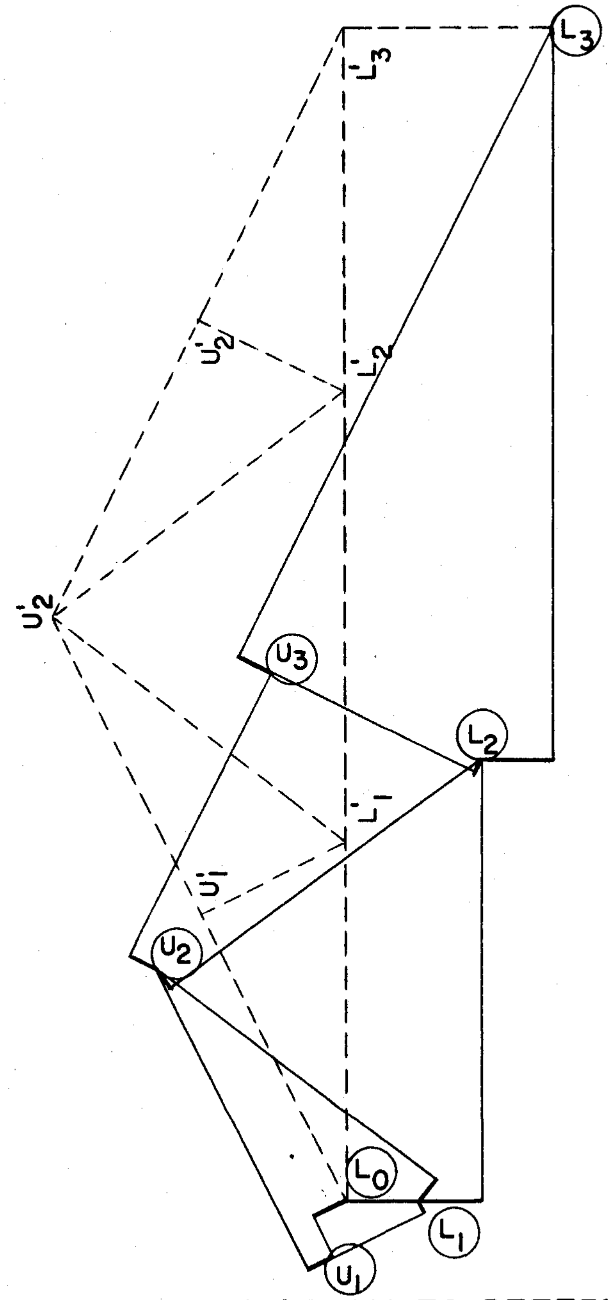
With these elongations and the use of the Williot diagram, the deflections of all panel points were determined, assuming L_0 to remain in position and the elongation of L_oL_1 fixed in direction (Figure 7). These values were checked by assuming member L_1L_2 fixed in direction (Figure 8).

With the deflections as shown in Table 8, the movement at each end of all members, at right angles to the members, was determined. The deflected truss is shown in Figure 9.

Calculating moments due to deflections

The moments causing the secondary stresses are a direct result of the calculated deflections. These fixed end moments

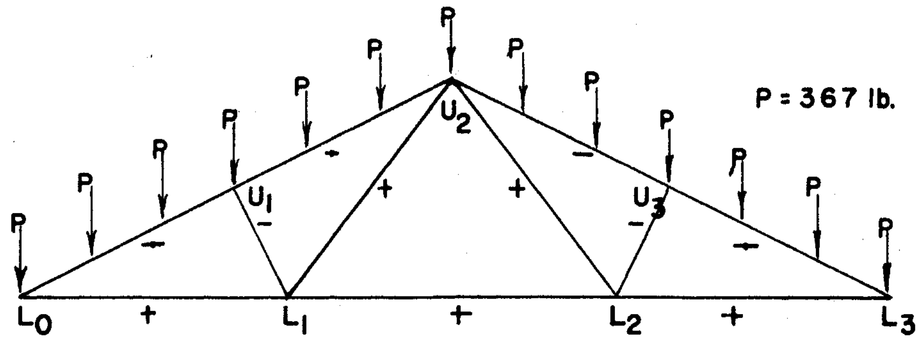




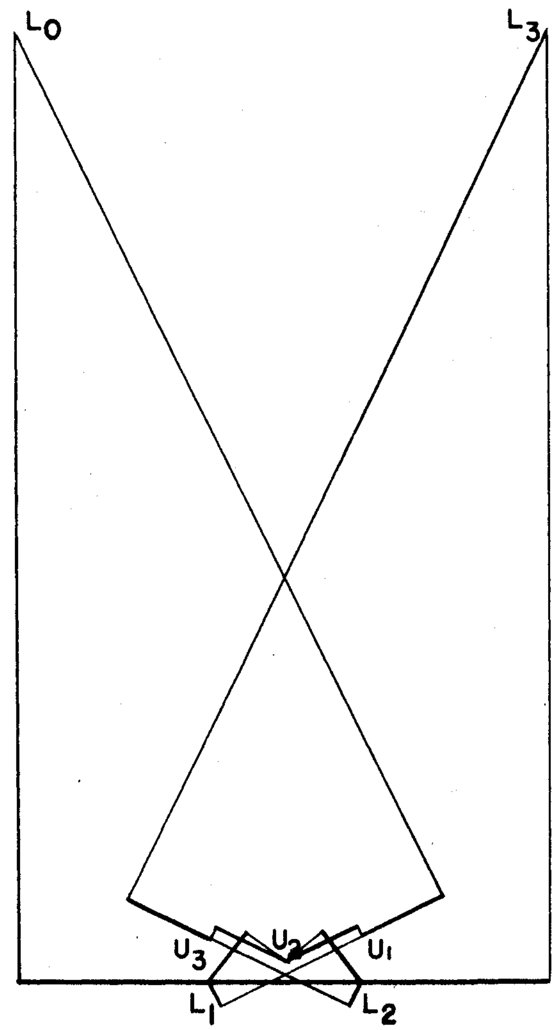
Scale: 1" = 0.1"
Point L_0 fixed.
L_0L_1 Fixed in direction.

joint	Deflection Hor.	Deflection Vert.
L_0	.000	.000
L_1	.039	.194
L_2	.071	.196
L_3	.109	.000
U_1	.069	.186
U_2	.054	.186
U_3	.039	.184

FIGURE 7 WILLIOT DIAGRAM TO DETERMINE DEFLECTION OF PANEL POINTS.



P = 367 lb.



Scale: 1" = 0.04"
Member $L_1 L_2$ assumed fixed in direction.

joint	Deflection Hor.	Deflection Vert.
L_0	.000	.000
L_1	.039	.197
L_2	.071	.197
L_3	.110	.000
U_1	.071	.188
U_2	.055	.193
U_3	.039	.188

FIGURE 8 WILLIOT DIAGRAM TO CHECK THE JOINT DEFLECTIONS.

were calculated by the following equation from Caughey (4):

$$M_{AB} = \frac{2EI}{L}\left(2\theta_B - \theta_A - 3R\right)$$

E = Modulus of elasticity 1,940,000 psi

I = Moment of inertia, inches^4

L = Length, inches

d = Deflection perpendicular to axis of member, inches

R = d/L

θ_B & θ_A = Angle between tangents of initial and final position.

In the present truss with glued joints, assuming no relative rotation between the members within the joint, both θ_B, and θ_A equal zero. Then the moment formula becomes:

$$M = \frac{2EI}{L} \times (-3R)$$

The sign notation for the moments is that proposed by Caughey (4), in which moments causing a clockwise resisting moment are positive and moments causing a counterclockwise resisting moment are negative. Table 9 lists the moments for each member calculated from the above formula.

Moment distribution

The calculated "fixed end moments" occur if the members act by themselves. Acting together in the truss, the moments are redistributed among members on the basis of the relative distribution factors of members which are proportional to their I/L values. The results of this redistribution are shown in Table 10.

Table 9

Moments Caused by Side Deflections of Members

	L in.	d in.	d/L=R	I/L=K	M* in.lbs.
L_0U_1	120	.200	.00167	.476	-9260
U_1U_2	120	.002	.00002	.476	+ 110
U_2U_3	120	.002	.00002	.476	+ 110
U_3L_3	120	.200	.00167	.476	+9260
L_0L_1	134	.198	.00148	.048	- 826
L_1L_2	165	.000	.0000	.039	0000
L_2L_3	134	.198	.00148	.048	+ 826
U_1L_1	60	.032	.00053	.06	- 654
U_2L_1	134	.010	.00007	.048	- 39
U_2L_2	134	.010	.00007	.048	+ 39
U_3L_2	60	.032	.00053	.106	+ 654

*$M_{AB} = \frac{EI}{L} (\theta_B + \theta_A - 3d/L)$. Assuming no rotation at the joints because of glue as a fastener θ_A and θ_B are equal to zero. E = 1,940,000. Then M_{AB} = 6 EKR

The summary of calculated stresses is shown with the measured stresses in Table 11.

Analysis of glue area at joint L_1

The strain in the members was measured at points as close to the gusset plates as possible; however, the moments were calculated from panel point deflections. These two points did not coincide and the distance between them differed from member to member. In order to calculate the twisting stress

Table 10

Moment Distribution for Moments Caused by Calculated Deflections in.lb.

	L_0		U_1			L_1			
	L_0U_1	L_0L_1	U_1L_0	U_1L_1	U_1U_2	L_1L_0	L_1U_1	L_1U_2	L_1L_2
D.F.	91	9	45	10	45	20	44	20	16
FEM	-9260	-826	-9260	-654	+ 110	-826	-654	- 39	00
Bal (1)	+9180	+906	+4412	+980	+4412	+304	+668	+304	+243
C.O.	+2206	+152	+4590	+334	00	+453	+490	00	-121
Bal (2)	-2140	-218	-2220	-484	-2220	-164	-362	-164	-132
C.O.	-1110	- 82	-1070	-181	00	-109	-242	00	+ 66
Bal (3)	+1085	+107	+ 563	+125	+ 563	+ 57	+125	+ 57	+ 46
C.O.	+ 281	+ 29	+ 542	+ 62	00	+ 53	+ 62	00	- 23
Bal (4)	- 282	- 28	- 272	- 60	- 272	- 18	- 40	- 18	- 16
C.O.	- 136	- 9	- 141	- 20	00	- 14	- 30	00	+ 8
Bal (5)	+ 132	+ 13	+ 72	+ 16	+ 73	+ 7	+ 16	+ 7	+ 6
C.O.	+ 36	+ 3	+ 66	+ 8	00	+ 6	+ 8	00	- 3
Bal (6)	- 35	- 4	- 33	- 7	- 34	- 2	- 5	- 2	- 2
C.O.	- 16	- 1	- 17	- 2	00	- 2	- 3	00	+ 1
Bal (7)	+ 15	+ 2	+ 8	+ 2	+ 9	00	+ 4	00	00
C.O.	+ 4	00	+ 7	+ 2	00	+ 1	+ 1	00	00
Bal (8)	- 4	00	- 4	- 1	- 4	00	- 2	00	00
C.O.	- 2	00	- 2	- 1	00	00	00	00	00
Bal (9)	+ 2	00	+ 2	00	+ 1	00	00	00	00
	- 44	+ 44	-2757	+119	+2638	-254	+ 36	+145	+ 73

Table 10 Continued

	U_2				L_2				U_3			L_3	
	U_2U_1	U_2L_1	U_2L_2	U_2U_3	L_2L_1	L_2U_2	L_2U_3	L_2L_3	U_3U_2	U_3L_2	U_3L_3	L_3U_2	L_3L_2
D.F.	45.5	4.5	4.5	45.5	16	20	44	20	45	10	45	91	9
FEM	+ 110	- 39	+ 39	- 110	00	+ 39	+654	+826	- 110	+654	+9260	+9260	+826
Bal (1)	00	00	00	00	-243	-304	-668	-304	-4412	-980	-4412	-9180	-906
C.O.	+2206	+152	-152	+2206	+121	00	-490	-453	00	-334	-4590	-2206	-152
Bal (2)	00	00	00	00	+132	+164	+362	+164	+2220	+484	+2220	+2140	+218
C.O.	-1110	- 82	+ 82	+1110	- 66	00	+242	+109	00	+181	+1070	+1110	+ 82
Bal (3)	00	00	00	00	- 46	- 57	-125	- 57	- 562	-125	- 564	-1085	-107
C.O.	+ 282	+ 29	- 28	- 281	+ 23	00	- 62	- 53	00	- 62	- 542	- 282	- 28
Bal (4)	00	00	00	00	+ 16	+ 18	+ 40	+ 18	+ 272	+ 60	+ 272	+ 282	+ 28
C.O.	- 136	- 9	+ 9	+ 136	- 8	00	+ 30	+ 14	00	+ 20	+ 141	+ 136	+ 9
Bal (5)	00	00	00	00	- 6	- 7	- 16	- 7	- 72	- 16	- 73	- 132	- 13
C.O.	+ 36	+ 3	- 3	- 36	+ 3	00	- 8	- 6	00	- 8	- 66	- 36	- 3
Bal (6)	00	00	00	00	+ 2	+ 2	+ 5	+ 2	+ 33	+ 7	+ 34	+ 35	+ 4
C.O.	- 17	- 1	+ 1	+ 16	- 1	00	+ 3	+ 2	00	+ 2	+ 17	+ 17	+ 1
Bal (7)	00	00	00	00	00	00	- 4	00	- 9	- 2	- 8	- 15	- 2
C.O.	+ 4	00	00	- 4	00	00	- 1	- 1	00	- 2	- 7	- 4	00
Bal (8)	00	00	00	00	00	00	+ 2	00	+ 4	+ 1	+ 4	+ 4	00
C.O.	- 2	00	00	+ 2	00	00	00	00	00	+ 2	+ 1	00	00
Bal (9)	00	00	00	00	00	00	00	00	- 1	00	- 2	00	00
	+1373	+ 52	- 52	-1373	- 73	-145	- 36	+254	-2638	-119	+2757	+ 44	- 44

Table 11

Summary of Calculated and Measured Stresses for the Fink Type Truss under Test

Member	Cross-section area in.^{2}	$= \frac{c}{I}$ in.^{3}	Axial load lb.	Moment from beam action in.lb.	Moment from deflection in.lb.
U_1L_0	12.2	.0656	-3700	+11,180	-2757
U_1L_1	5.9	.281	-1000	- 494	+ 119
U_1U_2	12.2	.0656	-3240	-10,686	+2638
U_2U_1	12.2	.0656	-3240	+ 7,606	+1373
U_2L_1	5.9	.281	+1100	- 24	+ 52
U_2L_2	5.9	.281	+1100	+ 24	- 52
U_2U_3	12.2	.0656	-3240	- 7,606	-1373
U_3U_2	12.2	.0656	-3240	+10,686	-2638
U_3L_2	5.9	.281	-1000	+ 494	- 119
U_3L_3	12.2	.0656	-3700	-11,180	+2757
L_0U_1	12.2	.0656	-3700	- 857	- 44
L_0L_1	5.9	.281	+3320	+ 857	+ 44
L_1L_0	5.9	.281	+3320	+ 392	- 254
L_1U_1	5.9	.281	-1000	- 326	+ 36
L_1U_2	5.9	.281	+1100	- 47	+ 145
L_1L_2	5.9	.281	+2200	- 19	+ 73
L_2L_1	5.9	.281	+2200	+ 19	- 73
L_2U_2	5.9	.281	+1100	+ 47	- 145
L_2U_3	5.9	.281	-1000	+ 326	- 36
L_2L_3	5.9	.281	+3320	- 392	+ 254
L_3L_2	5.9	.281	+3320	- 857	- 44
L_3U_3	12.2	.0656	-3700	+ 857	+ 44

Table 11 Continued

Moment	Calculated Stress: Axial psi	Calculated Stress: Bending psi	Calculated Stress: Deflection psi	Calculated Stress: Bending + axial psi	Experimental Stress: Axial[1] psi	Experimental Stress: Bending[2] psi	Experimental Stress: Deflect.[2] psi	Experimental Stress: Bending + axial psi	Defl.x100: Bending + axial psi
U_1L_0	-303	±733	±181	-1036	-333	±648	±160	-981	16.3
U_1L_1	-169	±139	± 33	- 308	-211	±139	± 33	-350	9.4
U_1U_2	-265	±700	±173	- 965	-334	±628	±101	-962	10.5
U_2U_1	-266	±500	± 90	- 766	-312	±296	± 54	-608	8.9
U_2L_1	+186	± 7	± 15	+ 193	+212	± 10	± 22	+222	10.0
U_2L_2	+186	± 7	± 15	+ 193	+101	± 38	± 81	+139	58.2
U_2U_3	-266	± 500	± 90	- 766	-297	± 263	± 47	-560	8.4
U_3U_2	-266	±700	±173	- 966	-288	±645	±160	-933	17.2
U_3L_2	-169	±139	± 33	- 308	-216	± 69	± 16	-285	5.6
U_3L_3	-303	±733	±181	-1036	-317	±518	±128	-835	15.3
L_0U_1	-303	± 56	± 3	- 359	-320	± 87	± 5	-407	1.2
L_0L_1	+565	±241	± 12	+ 805	+371	±318	± 16	+689	2.3
L_1L_0	+565	±110	± 71	+ 675	+394	±222	±104	+616	16.9
L_1U_1	-169	± 92	± 10	- 261	-232	±213	± 23	-445	5.1
L_1U_2	+187	± 13	± 40	+ 200	+139	± 43	±155	+182	85.0
L_1L_2	+373	± 5	± 20	+ 378	+427	± 8	± 34	+437	7.8
L_2L_1	+373	± 5	± 20	+ 378	+233	0	0	+233	0
L_2U_2	+186	± 13	± 41	+ 199	+142	± 30	± 56	+172	3.2
L_2U_3	-169	± 92	± 10	- 261	-210	± 44	± 5	-254	1.9
L_2L_3	+563	±110	± 71	+ 673	+463	±178	±113	+641	17.6
L_3L_2	+563	±240	± 12	+ 802	+447	± 49	± 17	+496	3.4
L_3U_3	-303	± 56	± 3	- 359	-310	±120	± 6	-420	1.4

[1] Average of stress on the four sides of each member

[2] Stress including bending plus deflection was measured as a combined stress and apportioned on the same ratio as calculated values.

Table 12

Measured Stress on the Surface of the Members Calculated by Multiplying Measured Strain by 1,940,000 psi

Member	Top[1]	Bottom[2]	Up side[3]	Down side[4]
L_1L_0	+466	+229	+415	+456
L_1U_1	-389	- 97	-200	-330
L_1U_2	+272	+ 48	+142	+ 97
L_1L_2	+378	+326	+359	+646
L_0U_1	-418	-233	-582	- 49
L_0L_1	+ 15	+683	+515	+272
L_3L_2	+ 62	+795	+155	+776
L_3U_3	-426	-175	- 78	-563
L_2L_1	+403	+402	+126	
L_2U_2	+107	+184	+165	+113
L_2U_3	-155	-233	-267	-190
L_2L_3	+504	+378	+427	+544
U_1L_0	+151	-825	-320	-340
U_1L_1	- 68	-275	-223	-272
U_1U_2	+145	-786	-326	-359
U_2U_1	+ 23	-680	-281	-310
U_2L_1	+233	+209	+165	+243
U_2L_2	+ 39	+136	+ 74	+155
U_2U_3	+ 29	-592	-330	-295
U_3U_2	+204	-766	-107	-485
U_3L_2	-120	-229	-194	-320
U_3L_3	+ 54	-726	-295	-290

[1]Top of member when laying down
[2]Bottom of member when laying down
[3]Top when truss is vertical
[4]Bottom when truss is vertical

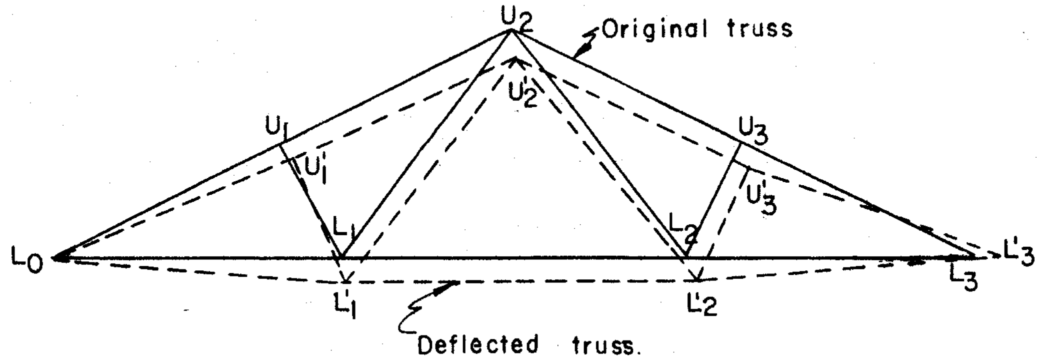


FIGURE 9 THE ORIGINAL TRUSS WITH THE DEFLECTED TRUSS SUPERIMPOSED IN DOTTED LINES.

on the glue joint it was necessary to determine the moment acting at the centroid of the area to be analyzed.

The diagram in Figure 10 shows the moment curve extrapolated to determine the twisting moment at the centroids of the two glue areas on either side of member L_1L_0 at joint L_1.

Distances A_1 and A_2 represent, to scale, the moments calculated from measured strains at the edge of gusset plates on joints L_0 and L_1 respectively. Distance B therefore represents the magnitude of anticipated moment at the centroid of the smaller glue area which failed. Distance C represents the magnitude of the anticipated moment at the centroid of the glue area on the opposite side of the joint.

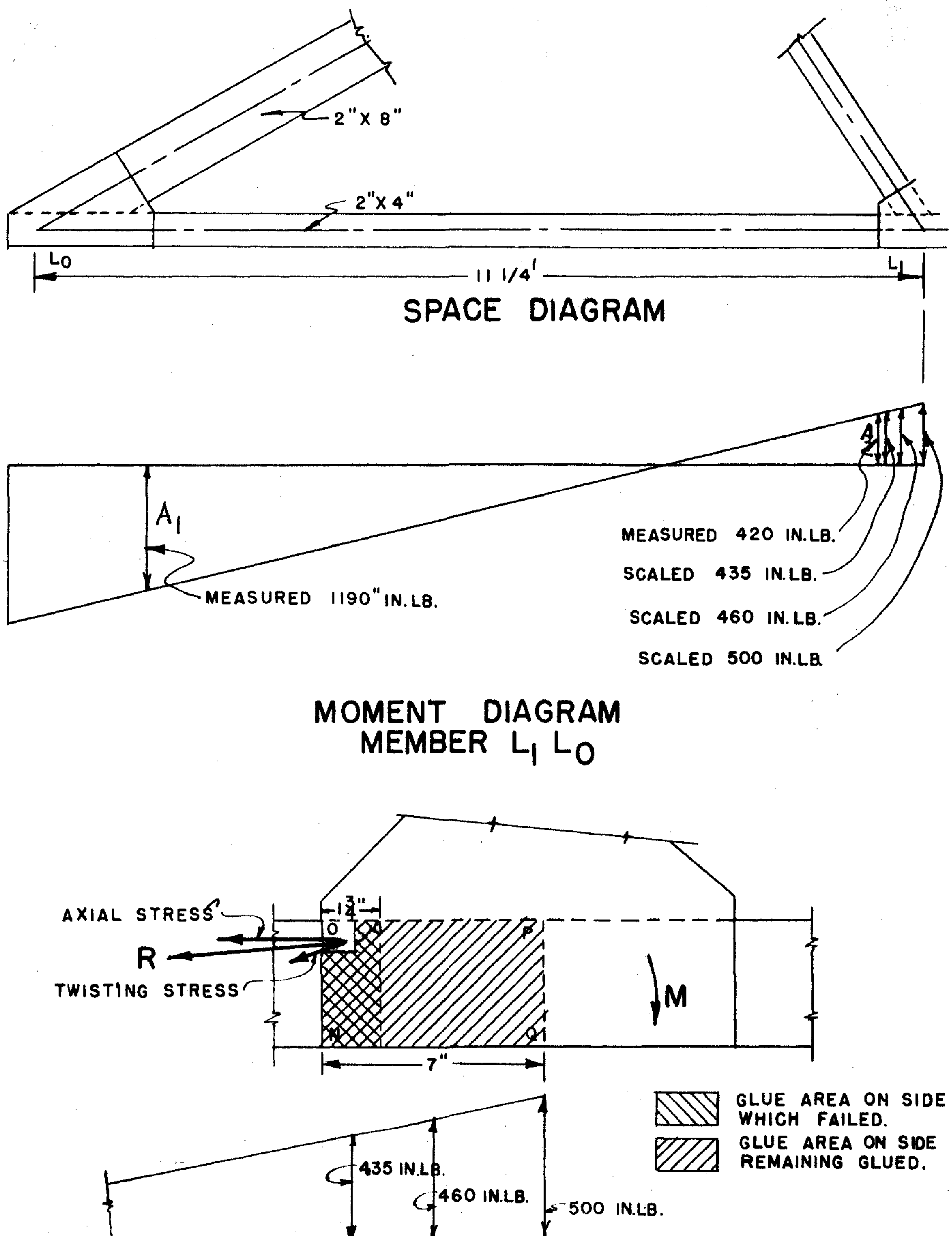


FIGURE 10 GLUE AREAS IN SHEAR, AND MOMENTS ON THESE AREAS AT JOINT L_1 OF MEMBER L_1L_O.

On each unit glue area there were two forces acting: (1) an axial force parallel to the axis of the members and (2) a twisting force perpendicular to a radius drawn from the centroid of the glue area in question. The total force acting at a point on the glue area would be the resultant of these two forces (See Figure 10).

Experimental Investigation

Materials and apparatus

Using hydraulic cylinders for loading the truss. In analyzing the reactions of structural members under load, a reduced scale model is used frequently. After surveying the possibility of using a model, it was thought that irregularities found in wood could not be reproduced in a model material. For this reason, a full scale truss was used.

In the literature, only one reference to testing full-scale wooden trusses explained a loading mechanism. In this experiment (23), the truss, standing in a vertical position, was loaded by adding sand to a flat platform suspended below the truss from the upper panel points. Since the test was long-term with an overload, the truss was loaded and allowed to stand.

This arrangement was impractical for the purpose of the present project as the load had to be applied and removed a number of times. For this reason, some loading apparatus was needed which would easily and quickly apply the load, and still maintain its accuracy.

Several possibilities were considered:

1. The use of lead weights on steel rods similar to the weights on a balance scale.
2. The use of water in barrels suspended from the top panel points.
3. The use of sand bags piled on the top chords.
4. The use of a system of pulleys attached at the points of loading.
5. The use of hydraulic cylinders.

The first four possibilities have obvious disadvantages where the load must be applied and removed a number of times without employing excessive time and labor. The fifth possibility appeared to have considerable merit. A large number of equal loads could be applied with good accuracy and the eleven required equal loads could be applied with one hydraulic pump.

The eleven cylinders were calibrated by attaching the cylinder to the loading end of a balance scale and recording the pull on the cylinder for various fluid pressures. Figure 11 shows the mechanism, while the results are shown in Appendix A.

These data show very small tolerances between cylinders in producing equal loads at a given pressure. The results obtained from the progressive loading on cylinders number one, two, and three are plotted in Figure 12. It was found that the pressure was proportional to the load with small variations. These results suggested that the remaining cylinders would have

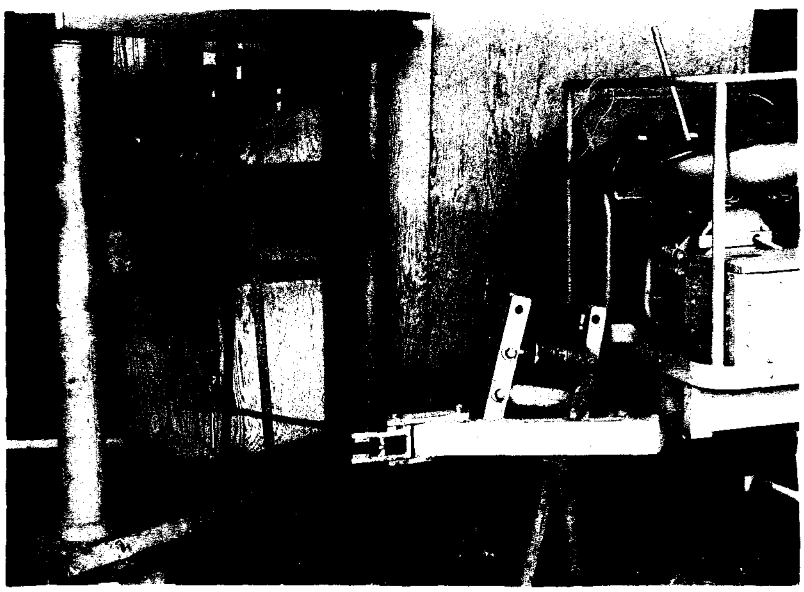


Figure 11. Hydraulic pump and gages used to test the consistency of the hydraulic cylinders

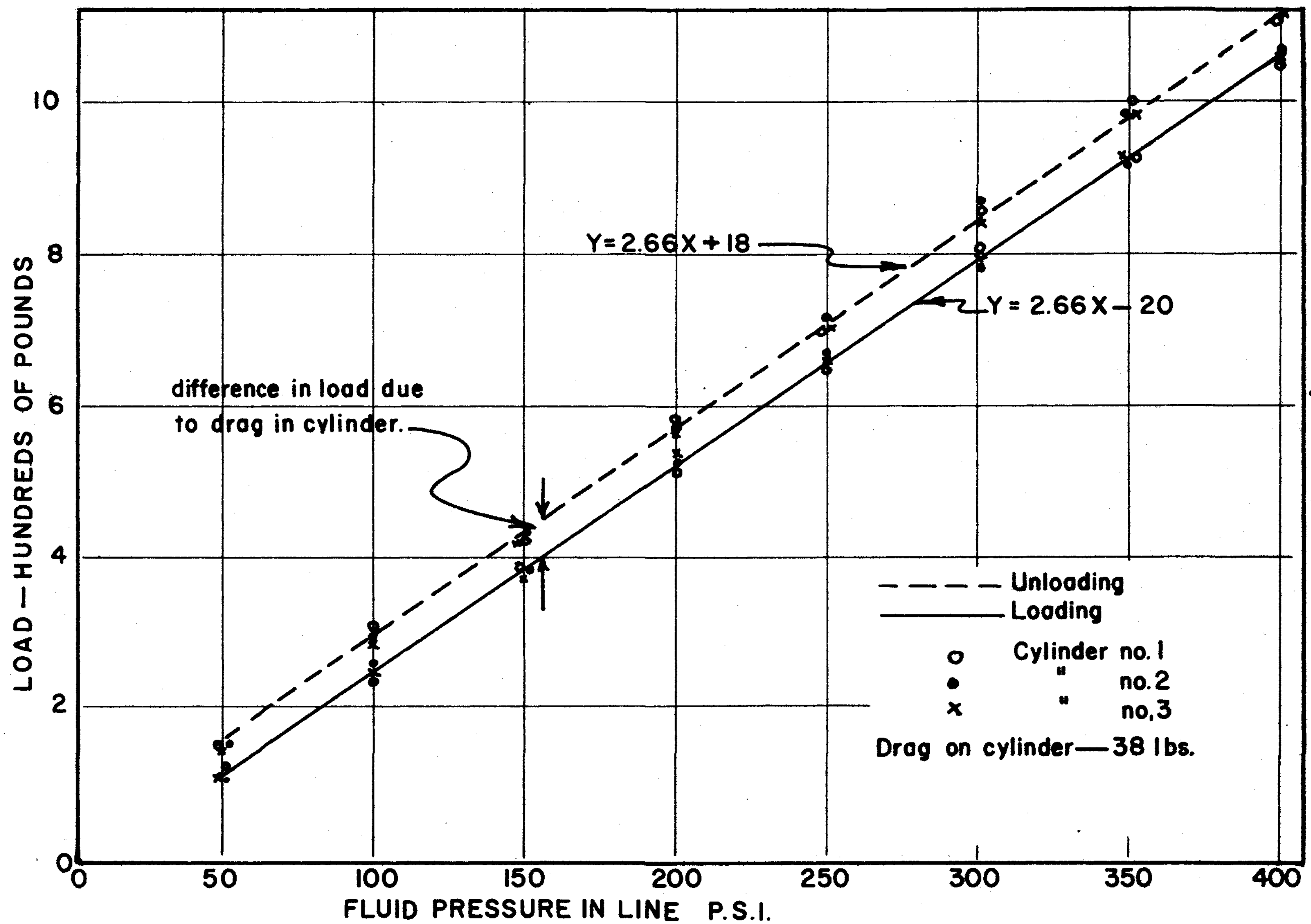


FIGURE 12 CALIBRATION CURVE CYLINDERS 1, 2, & 3

similar characteristics. When the cylinders were unloaded, a constant drag of 38 pounds was observed on all the cylinders. This should be considered in running future tests.

The close tolerance and consistency between the cylinders make them suitable for applying equal loads to the truss. The speed of loading can be governed by the time required to record the data.

Instrumentation. The basic unit for measuring strain was the SR-4 model A-1 strain gage having a gage length of 15/16 inch and a gage factor of 2.06. A strain rosette, which consists of a group of three strain gages oriented at fixed known angles to each other, was used on the gusset plates. A Baldwin Southwark Model "K" balancing unit with a D-C battery power supply was used to measure the strain produced on one-half of the truss. A Young Testing Laboratory balancing unit was used to measure the strains on the other half of the truss.

Single strain gages were placed on the four surfaces at each end of all the members and strain rosettes on a gusset plate at joints L_0, L_1, L_2, and L_3. A total of 110 individual gages were used. Since the "K" recorder can only read one gage at a time, a system of switches was constructed (Figure 13) to switch quickly from one strain gage to the next. With this set-up all of the strain gages could be read from a central point. A wiring diagram of the selector switch is shown in Figure 14.

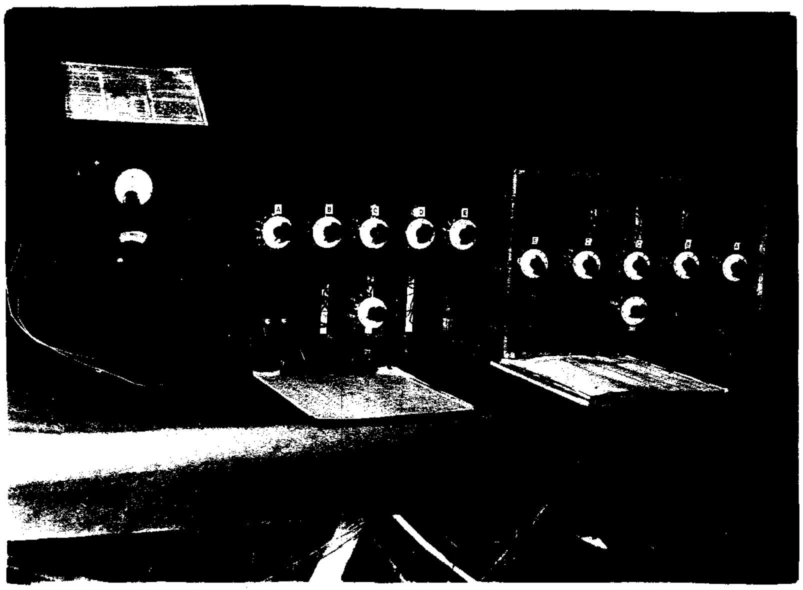


Figure 13. Switching boxes used to switch between SR-4 gage and Model K recorder

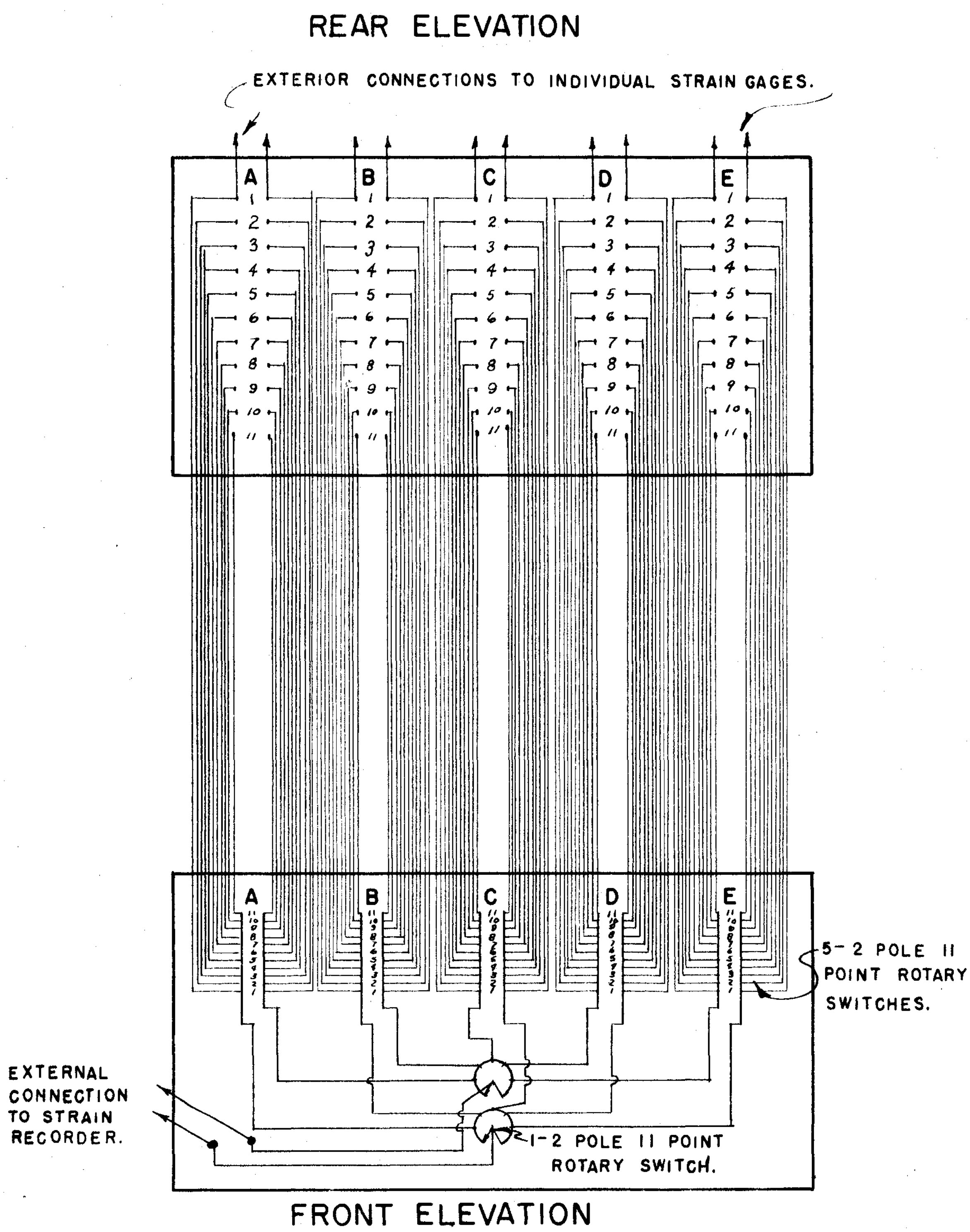


FIGURE 14 WIRING DIAGRAM OF SELECTOR SWITCH.

In order to measure the deflections of the panel points, seven Federal dial indicators and six Ames dial gages with a least count of one-thousandth of an inch were used. A standard No. 12 wood screw two inches long with a one-inch steel cube welded to the head was screwed into the truss at the panel points (Figure 15). The plungers from the dial gages rested on the surfaces of the cube to measure the motion in two directions.

Determining the location of maximum stress. The brittle lacquer used for determining the location of maximum stress was that sold under the trade name "Stresscoat", and the application was made under the direction of Mr. Othmer of the Theoretical and Applied Mechanics Department. A regulation kit including a spray gun for the aluminum base coat, a spray gun for the lacquer, and an assortment of formulas for different atmospheric conditions was used (Figure 16).

Constructing the testing frame. The truss rested in a horizontal position on a floor consisting of 2" x 4" joists with spacings varying from 4 feet to 6 feet. This variable spacing was used in order to prevent the gusset plates from resting on a joist. Since the frame was located in a dirt floor building, a layer of roofing paper was placed over the surface of the ground before the joists were placed to protect the strain gages on the lower side of the truss from moisture. The top surfaces of the joists were coated with paraffin wherever a truss member and a joist made contact. The paraffin reduced any friction which might affect the load.

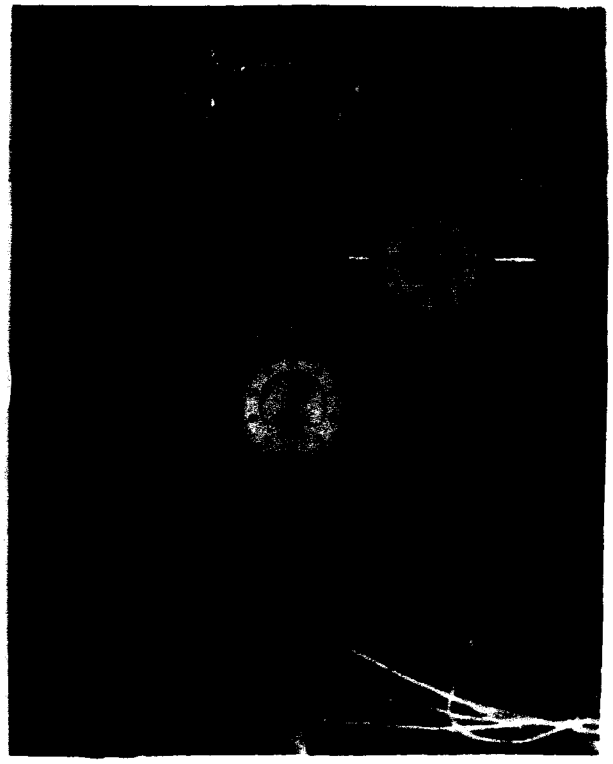


Figure 15. Dial gages for measuring panel point deflections bearing on metal block screwed into panel point

Figure 16. Stresscoat equipment used to apply brittle lacquer to the joints.

The most economical and practical resisting structure appeared to be a 4" x 8" beam formed by spiking together one 2" x 8" plank and two 1" x 8" boards. This 36-foot beam was continuous over five supports. The diagram of the truss in the testing frame is shown in Figures 17 and 18.

Loading apparatus. Eleven hydraulic cylinders with the following specifications were used: Bore 2 inches, stroke 6 3/4 inches, shaft 5/8 inch diameter, length 10 5/8 inches (Figure 19a). The piston area on the shaft side of the piston was 2.66 square inches. One end of each piston was connected at the loading point on the top chord by means of a 1/8" x 1" iron strap as shown in Figure 18a. The other end was fastened to the 4" x 8" wooden beam as shown.

Oil was pumped to the cylinders through a 1/4 inch copper tube with a 3000 psi hydraulic pump. To be sure the pressure on all of the cylinders was the same, pressure gages were installed at each end of the line, so the pressure could be checked continually.

Constructing the test truss. No. 1 grade Douglas fir was used in the truss. Individual boards were selected to obtain material as near the same density as possible. When the truss was built, the 2" x 4" members had the same density, although the densities of 2" x 8" members were slightly higher (Appendix D).

The moisture content of each member was measured to be sure it was below 20 per cent -- a safe moisture content for good glue contact.

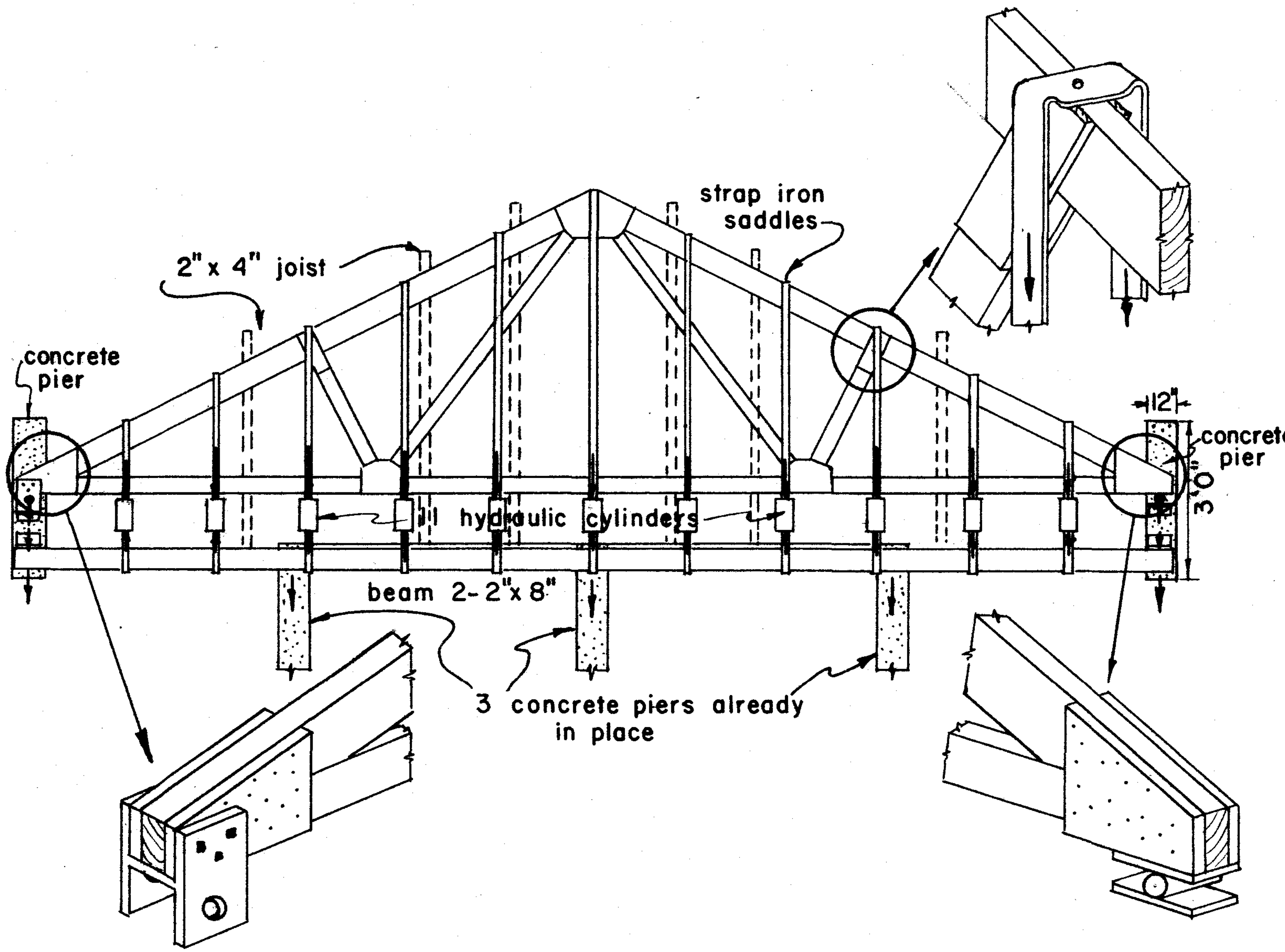
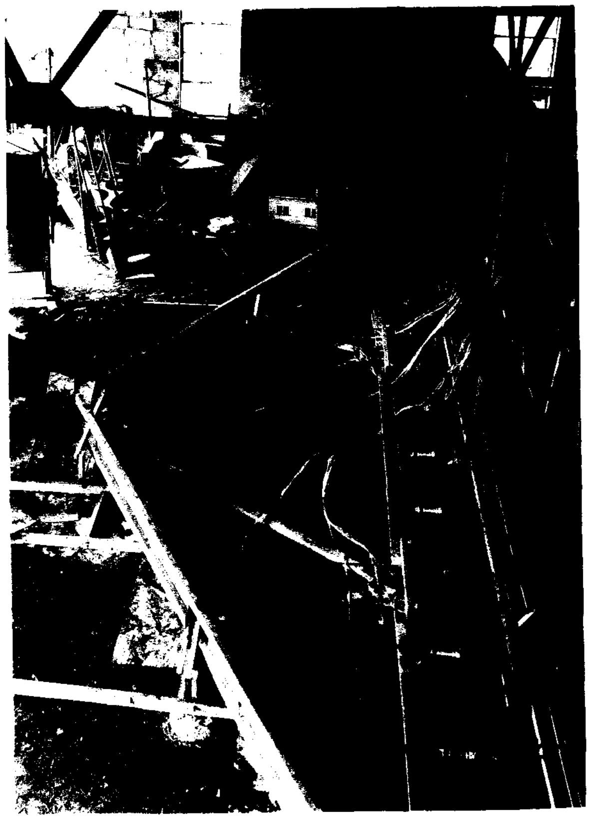
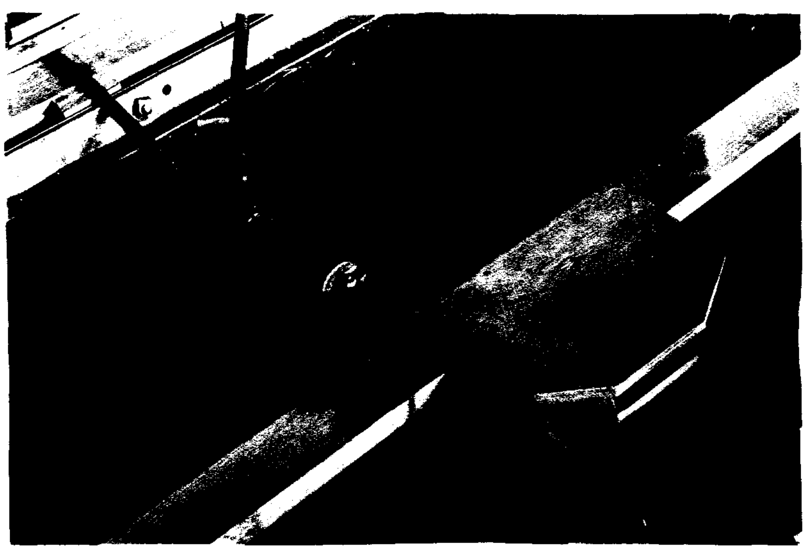


FIGURE 17 APPARATUS FOR LOADING TRUSS

Figure 18. Apparatus for applying load to the full scale truss



(a)



(b)

Figure 18. (a) View of the truss ready for testing with all gages in place and roofing section on top chords
(b) View of hydraulic cylinder connected between truss saddles and resisting beam

Figure 19. The hydraulic system for loading the truss

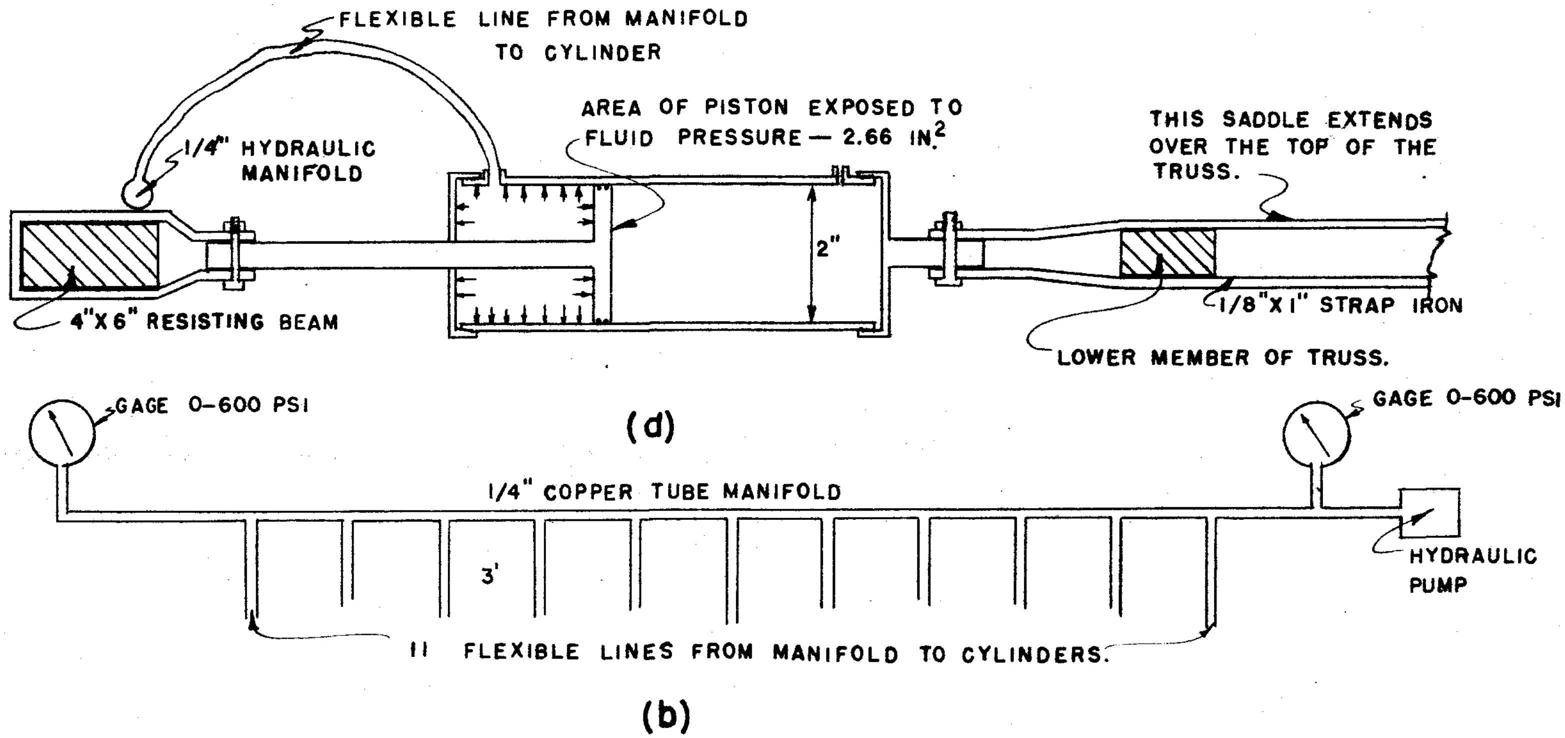


FIGURE 19 (a) SECTION OF HYDRAULIC CYLINDER AND CONNECTING MECHANISM. (b) SCHEMATIC LAYOUT OF HYDRAULIC SYSTEM.

Powdered casein glue was mixed according to directions with equal parts by volume of powdered glue and cold water. An aging period of 45 minutes was allowed before the glue was applied to the wood.

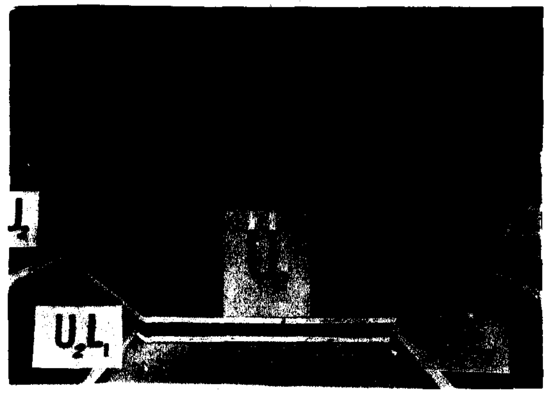
Gusset plates were made from 3/4-inch grade A-A exterior fir plywood. They were cut so that the edge of the gusset plate crossed the axis of all truss members at 90°. This accounts for the irregular shape of the gusset plates. Under actual construction conditions, the shape could be changed to utilize the plywood more effectively.

In the first truss, glue was applied to the surface of the member and the gusset plate. Pressure was applied with 8d nails spaced 3 to 4 inches apart. The final truss was assembled with 8d scaffold nails at the same spacing (Figure 20). In some joints, this spacing varied slightly, due to the irregular shape of the members. After the glue set for 48 hours, the nails were removed, leaving the glue as a fastener. Then the truss was turned over and the process of gluing the gusset plates was repeated.

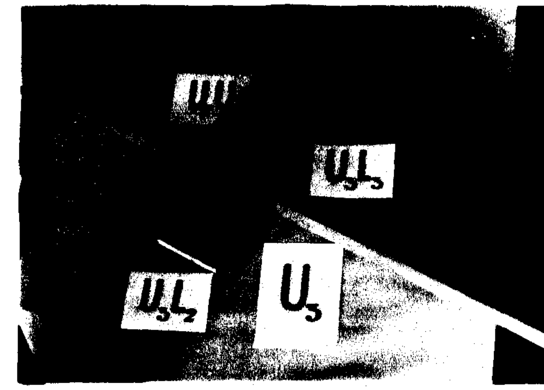
After the truss was put in the testing frame, and the loading straps were placed, a section of roof deck 16 inches wide was nailed to the top chord to provide stability, similar to that found under service conditions (Figure 18a).

Determining location of maximum stress in preliminary truss

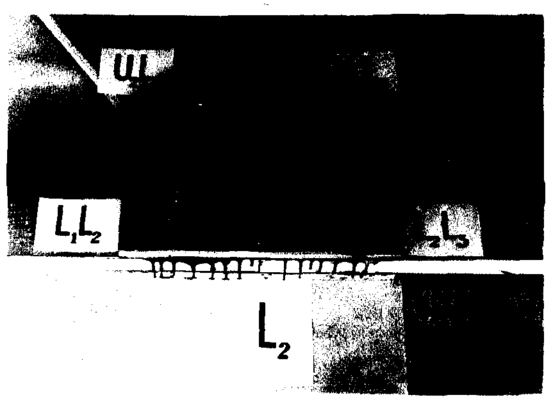
The literature indicated a brittle lacquer had been used



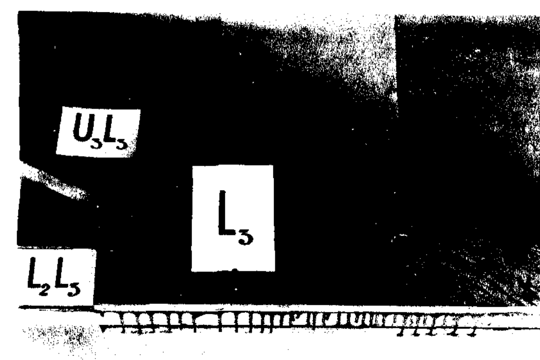
(a) Joint U_2



(b) Joint U_3 similar to U_1



(c) Joint L_2 similar to L_1



(d) Joint L_3 similar to L_0

Figure 20. Gusset plates glued and nailed with 8d scaffold nails

to measure stress in metallic members. Since this material had not been used on wood, the following preliminary tests were conducted to determine its suitability.

A wooden beam 1 3/4" x 1 3/4" was coated in the laboratory. Measuring load applied and beam dimensions provided a check against the strain indicated by the lacquer. In one test, the value of stress recorded by the lacquer was 13.6 per cent higher than the calculated value, while in the second test it was 12.1 per cent lower than the calculated value. These tests showed that the lacquer could be used as a good indication, qualitatively, of the location of maximum strain as the cracks developed first where the strain was highest. They also were perpendicular to the direction of the principal strain.

The preliminary truss was used to check the loading apparatus and the procedure for testing.

To observe the operation of the equipment, fluid was pumped into the line until the design dead-load-plus-snow was placed on the truss. When all parts of the system were checked, the load was removed and preparations were made for applying the brittle lacquer.

At 6:00 AM, the aluminum undercoat and the lacquer (formula 1211) were applied at a temperature of 70° Fahrenheit. Curing took place at 70° for several hours. This temperature was lower than that recommended by the manufacturer. However, it was anticipated that the correct temperature for testing would be reached by 4:00 PM. In the afternoon, the temperature and humidity corresponded to that recommended for testing. The hydraulic cylinders were energized, and the load was applied at the rate of 33 pounds per minute until a load equal to two and one-half times the design load had been added. The strain sensitivity of the lacquer was less than that in the members; however, during the curing process the lacquer crazed, which made it difficult to observe any strain cracks. The failure of the lacquer to reveal strain cracks was attributed to an elasticity caused by the temperature (70°F) at which the lacquer had been cured: the creep allowed the strain deformation in the lacquer to keep up with that of the member.

The next day this coat of lacquer was scraped off and another of formula 1210 applied. During the curing of this coat, heat lamps were rigged over each joint to maintain a temperature of 100°F on the surface of the gusset plate for one hour. Then the temperature was reduced to 90°F for three hours, and kept between 85°F and 90°F for twenty hours.

When the lights were removed, a temperature of 83°F and a relative humidity of 40 per cent, recommended by the manufacturer as desirable for formula 1210, prevailed. When the truss was loaded to 750 pounds per cylinder, cracks occurred as shown in Figure 21.

At the same time as the cracks appeared, member L_0L_1 pulled loose at joint L_1. Figure 22 shows the failed joint

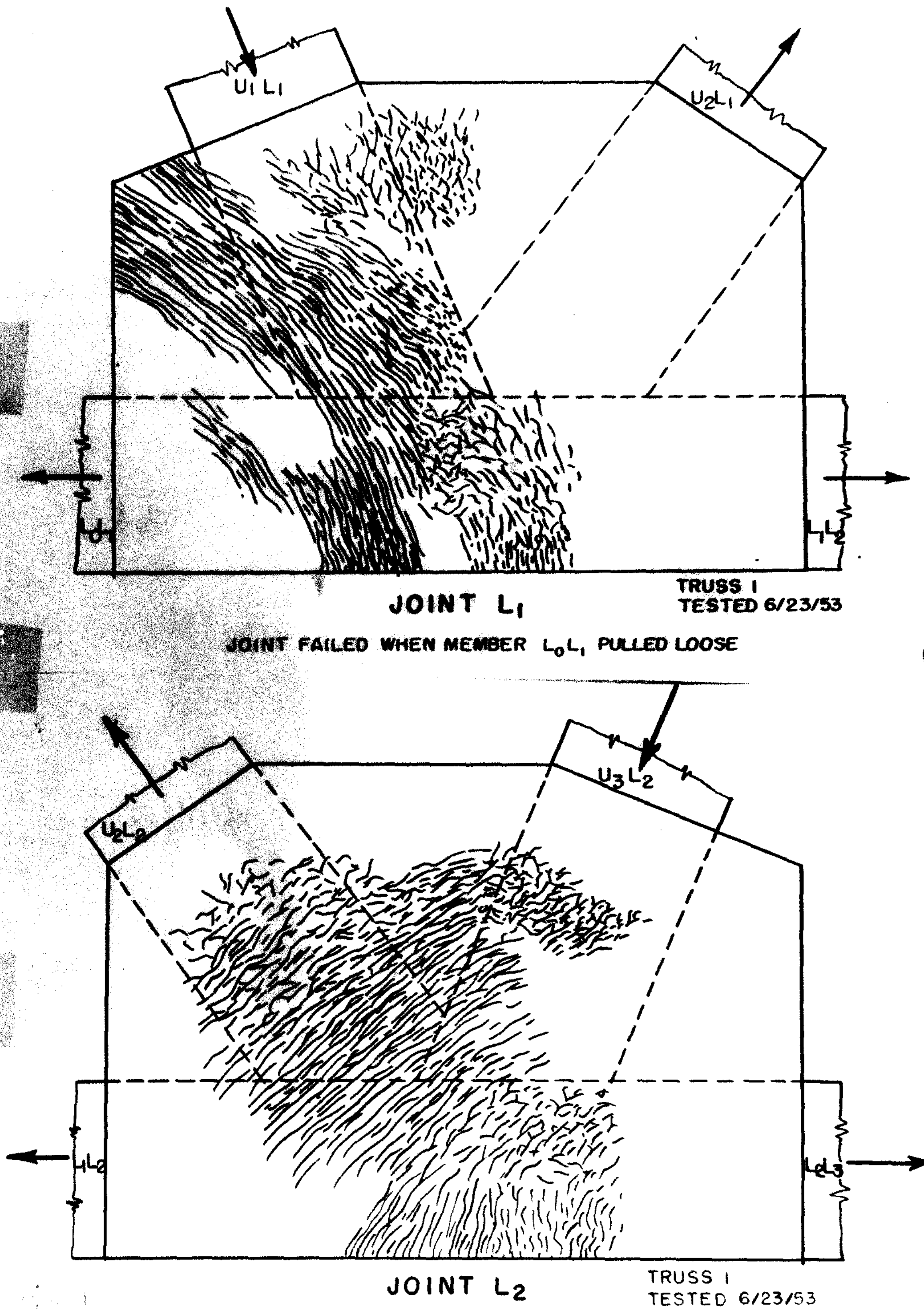


Figure 21. Cracks appearing in stresscoat when truss was loaded

and indicates that the cause of failure was a starved glue joint between the gusset plate and member L_0L_1.

Investigating the cause revealed that member L_1L_2 was approximately 1/16 inch thicker than the others in the joint and, when the gusset plate was nailed, it did not make contact with the other three members close to the end of the thick member. This is shown in Figure 22 by the dark areas at the ends of members U_1L_1, U_2L_1, and L_0L_1. Where failure did occur it was in the wood fibers and not in the glue.

Figure 21 shows a sketch of the strain cracks as they appeared at joint L_2 at the same time that member L_0L_1 failed. These two gusset plates should be mirror images of each other; however they are not. At joint L_2 the concentration of strain occurs at the intersection of the axis of the members, while at joint L_1 the center of strain has shifted toward the point of failure. Where good glue contact is obtained, the unit strain in the gusset plates would be less, with a lower resultant unit stress.

Final truss

Applying brittle lacquer. At 8:30 AM, July 27, Stresscoat (formula 1210) was applied to the second truss with the temperature at 76°F and the relative humidity at 82 per cent. Heat lamps were rigged over the joints to maintain a temperature of 108°F, until 5:00 PM. At this time the temperature was lowered to 96°F, and maintained at that level until time for testing.

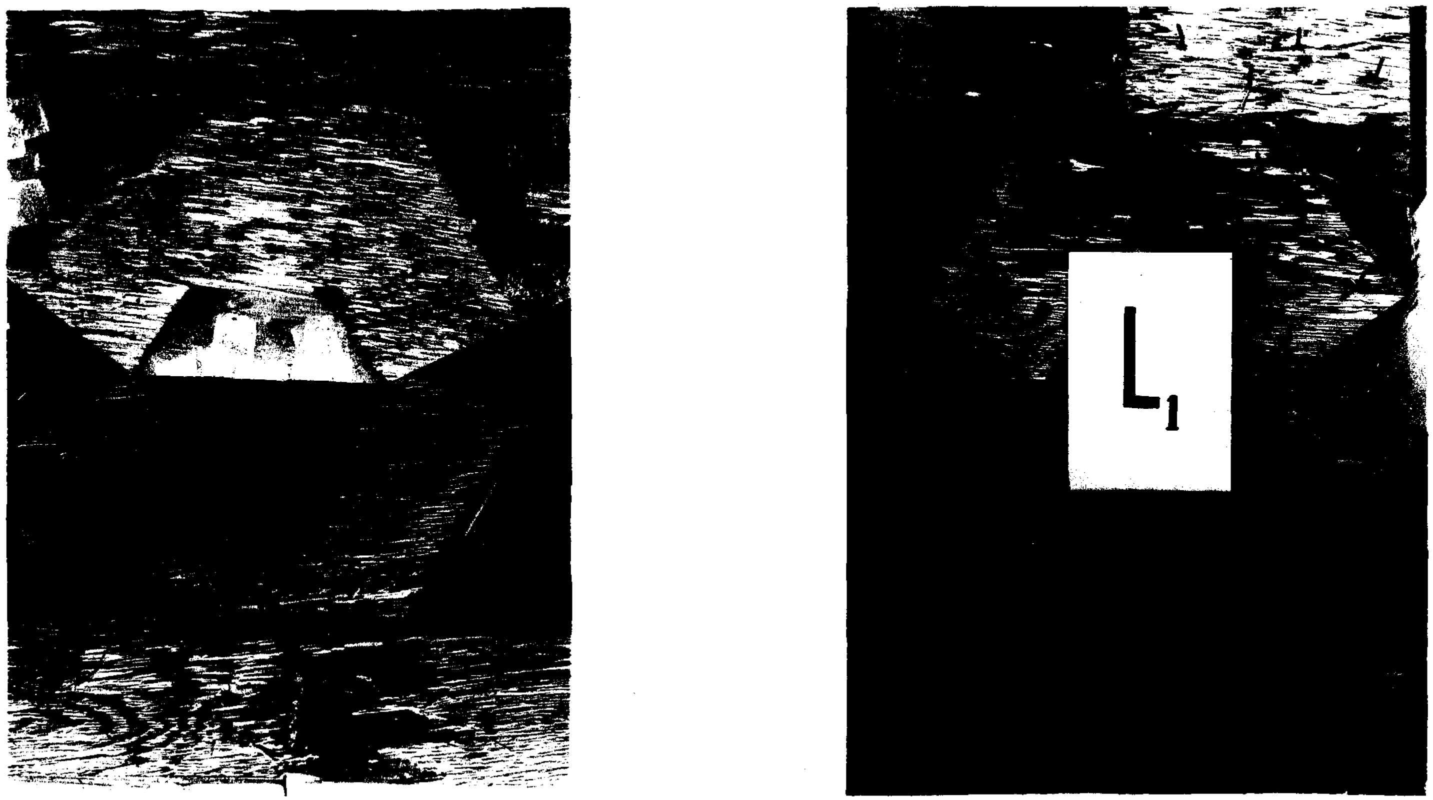


Figure 22. Gusset plates on joint L_1 after failure occurred. Glue starved area at bottom of right picture.

At 8:00 AM, July 28, the lights were turned off and the lacquer temperature was cooled to 90°F while the ambient air temperature was 82°F. This temperature differential of 8° between the lacquer and the air tended to make the lacquer more sensitive. At this same time the truss was loaded at the rate of 133 pounds per point per minute until a load of 660 pounds per point was added. Figure 23 shows the location of the cracks which appeared.

There were no cracks in joints L_0 and L_1, for the lacquer on these joints was too thick, This was evidenced by the fact that several small cracks appeared at the edge but disappeared very soon as they progressed toward the center of the area.

Comparing Figure 21 and Figure 23 for joint L_2 shows that the cracks appeared at the same angle and were concentrated in the same area. It was assumed that the mirror image of this joint would be that of joint L_1. Cracks perpendicular to the long tension member showed that that member L_1U_2 might be causing the major stress in the gusset plate.

Locating and applying the strain gages. In order to analyze the effects of secondary stresses, the unit stress at the ends of each member must be known. The axial stress is the average of the stresses on four sides of rectangular members, and the bending stress is one-half the difference between the plus and minus stress on the top and bottom of the member (18). Strain gages for measuring these stresses were located on the center line of each side of the member,

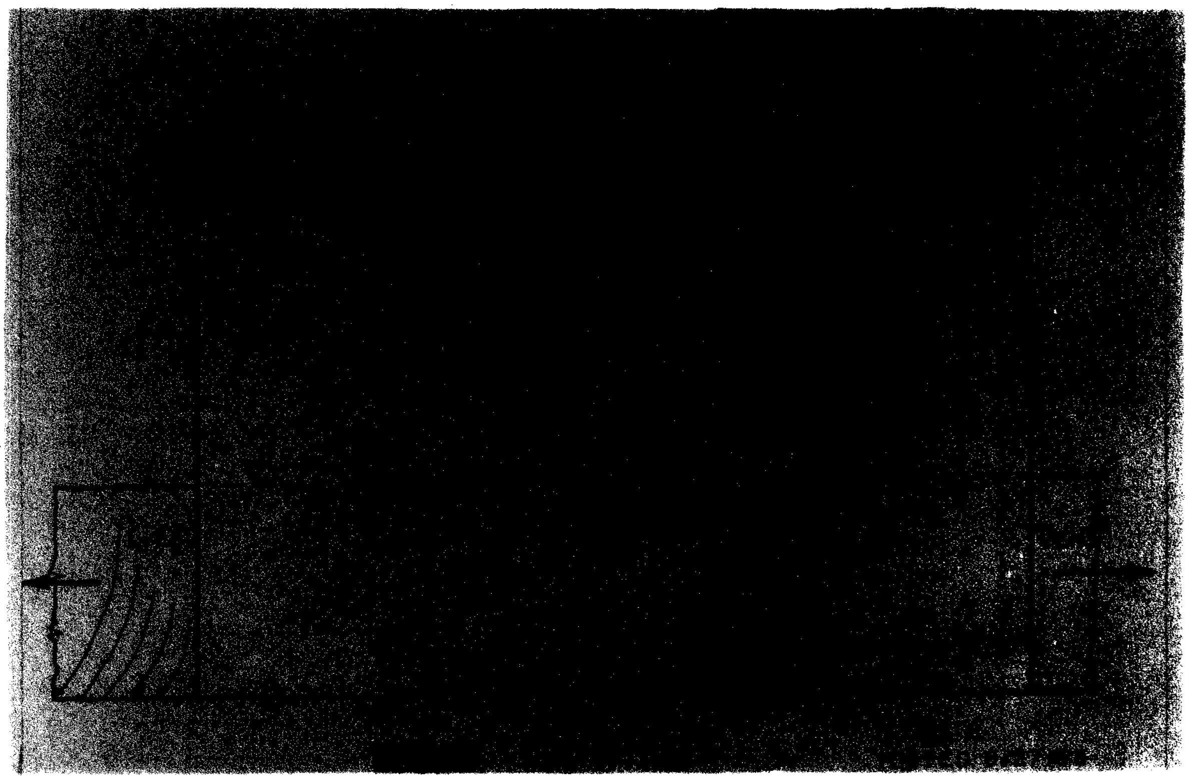


Figure 23. Cracks appearing in Stresscoat on joint L_2 when truss was loaded

as close to the edge of the gusset plate as possible (Figure 24).

The strain rosettes were located on the gusset plates where the strain cracks first appeared, and were closest together. One gage of the rosette was oriented at right angles to the direction of the cracks, one parallel to the cracks, and the third oblique to the other two. With these three values of strain, the effect of the Poisson ratio could be considered in the determination of stresses.

The surface of the member was sanded to a smooth finish, coated with a film of Duco cement, and allowed to dry. A second coat of cement was applied to the member; at the same time a coat was applied to the strain gage which was placed on the member. Very slight pressure was applied to the gage while the cement was drying. When thoroughly dried, the gage was coated with a layer of wax to protect it from dampness.

Leads of No. 18 copper wire were soldered to the terminals of each gage and run to the selector switches. This made it possible to switch quickly from gage to gage without having to change the balancing unit. Where convenient, one common lead was used, to decrease the number of wires involved. The wiring code shown in Appendix B was used in making connections to the switch box. Two similar circuit systems were used -- one for the left half of the truss (Switch 1) and the other for the right half of the truss (Switch 2). This arrangement was used so that the balancing instruments could be used without

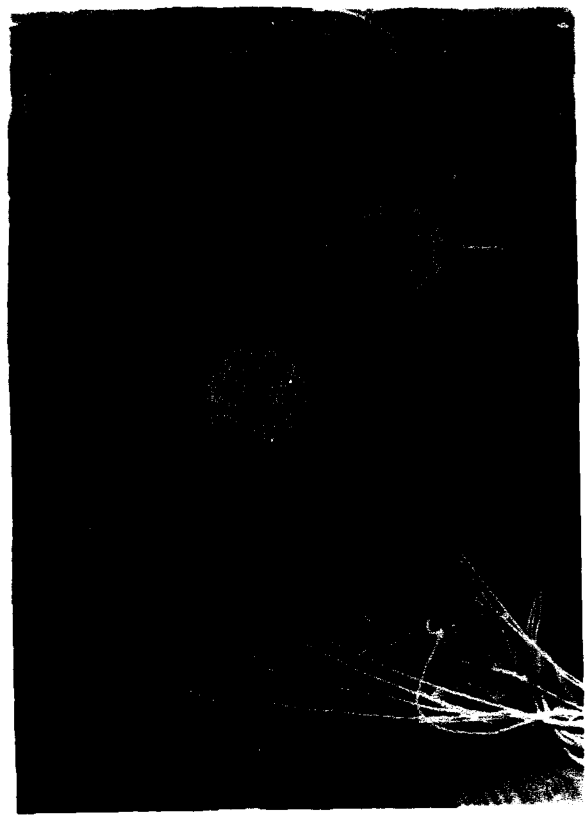


Figure 24. Panel point U_1 showing the strain gages in position and wired to selector switch

changing the wire connections, for it had been found that changing leads between the switch panels and the balancing instruments caused errors in the readings.

When all the circuits were completed, a check was made, by tapping the strain gage with the finger, to be sure the gages were operating. If the gage was operating it would fluctuate, but return to zero. Gage D-7 on switch 2 was the only gage not functioning properly. It was located on the bottom side of the truss, where it could not be replaced.

Running the test. A load of 133 pounds, represented by a fluid pressure of 50 pounds per square inch, was the increment of loading. After the load was applied, and the oil pressure became constant, one person recorded the readings of the deflection gages and two persons recorded the strain readings. The pressure in the fluid was kept at a constant level to keep a constant load on the truss.

It required thirty minutes to take all the necessary readings, so the load was applied at the rate of 133 pounds every thirty minutes up to a total load per point of 532 pounds. The load was then removed, and the truss allowed to assume its original shape.

After the first run, the truss did not return to its original position. Investigation showed that the right reaction slipped. After the reactions were repaired, and reinforced, the procedure for loading was repeated three times.

Results

The strains for each increment of load for each strain measuring point were plotted independently to determine the strain for the design load. For all gages except the two A-8 gages (Appendix C), this curve was represented by a straight line. The locations of the two A-8 gages, on the top side of the members L_0U_1 and L_3U_3 were the only two points where the strain did not follow a straight line relationship. Here the strain increased until a load of 150 psi fluid pressure was applied, and then decreased, the gage at L_0 changing from positive to negative. At all other points, the strain increased constantly in either a positive or negative direction once that direction was determined. The strains at the design load of 137 psi fluid pressure were then multiplied by the modulus of elasticity (E) of 1,940,000 psi to obtain the unit stress at the design load. These unit stresses are recorded for each surface in Table 10.

A summary of the calculated and experimental values is shown in Table 11.

The use of glue with plywood gusset plates for assembly presents a very practical method for prefabricating timber trusses. These prefabricated units are useful on farms where inexperienced labor constructs the building. One difficulty encountered in this investigation was that of obtaining a good glue area when using common dimension material. When one member under a gusset is slightly thicker than the others, the result is a starved glue joint.

Figure 22 shows a starved glue area at joint L_1 of the preliminary truss. Member L_1L_0 had a full area of 25.4 square inches on one gusset, but only 6.34 square inches on the opposite gusset. Each segment of these glue areas must resist both a twisting and an axial force due to the loading. The twisting moment at the member ends was measured at the edge of the gusset plate; thus the moment was different at the center of rotation of each glue area as shown in Figure 10.

The glue areas on each gusset resisted one-half of the twisting moment at the end of the member. Under this assumption, the stress on the glue area at the design load of 137 psi fluid pressure was as follows:

For the starved glue area

Twisting $$S = \frac{Mc}{I_p} = \frac{217 \times 2.01}{8.55} = 50.3 \text{ psi}$$

M = 1/2 the twisting moment at the centroid of the area or $\frac{435}{2}$ in.lb. as shown in Figure 10.

c = Distance from centroid of area to farthest point in inches.

I_p = Polar moment of inertia in inches^4.

Axial $$S = \frac{P}{A} = \frac{3320}{2} \times \frac{1}{6.34} = 262 \text{ psi}$$

P = Axial load in pounds

A = Cross section area of member in inches^2.

and the resultant $S = 307$ psi (by graphics)

For the area with full glue contact

Twisting $S = \frac{Mc}{I_p} = \frac{230 \times 3.9}{131.2} = 6.7$ psi

M = Moment on centroid of glue area $\frac{460}{2}$ = 230 in.lb. as shown in Figure 10.

c = Distance from centroid to farthest point in inches.

I_p = Polar moment of inertia of glue area in inches^4.

Axial $S = \frac{3320}{2} \times \frac{1}{25.4} = 65.5$ psi

and the resultant $S = 68$ psi (by graphics).

DISCUSSION OF RESULTS

The measurement of strain in wood presents several problems not common to most other materials. Due to its non-homogeneity, the strain in the individual parts of the material varies under stress. It becomes necessary, then, to measure an average strain over a small section of the member. This was done with the SR-4 bonded strain gage. The strains recorded in this study were measured over a definite area and the character of the wood directly under the SR-4 strain gage could have influenced the values recorded.

From the recorded data it can be seen that strain gages give reasonably consistent readings. When the strains for individual points were plotted against applied load, the curves were straight lines with small variation between points. Professor Baten, the statistician for the Michigan State College Experiment Station, suggests that a straight line is the most representative for each set of values. By taking the value of strain at the design load from these curves any local difference in the recorded value would not affect the strain to be used in the calculations.

The stresses due to axial loads, to bending introduced by loading in top chords, and to deflections of truss members were calculated for the end of each member. Under ordinary design conditions the first two of these influences are taken into account and in this discussion are considered to be primary stresses. The third influence plus other defects in construction is not normally allowed for in computations and is considered, therefore, as a secondary stress.

The measured axial strains were obtained by averaging the strains on the four surfaces of each member. The strains caused by bending were measured as a combined strain which included both the effects of bending in the top chords and the effects caused by truss deflection. These two were separated according to the ratio between the corresponding calculated values. The stresses calculated from these strains were then regrouped into primary and secondary stresses as previously described. These strains were multiplied by the modulus of elasticity to determine unit stress.

The measured axial stresses were uniform throughout the members of the truss, and agreed reasonably well with the calculated values. In a discussion of the results with the Forest Products Laboratory personnel, they indicated that results might not be consistent when measuring these small strains. The SR-4 gage is accurate to within 10 per cent. Thus with these limitations, the measured values agree with the calculated values as indicated in Table 11.

Stresses caused by bending in the top chords are high in most members and, when combined with the axial stresses, represent a large proportion of the total stress.

Secondary stresses, the result of truss deflection, comprise a small percentage of total stress as shown in Figure 11. In member U_2L_2, secondary stresses were 58.2 per cent of primaries and in member L_1U_2, they were 85.0 per cent, while in all the other members they were 17.6 per cent or less. The fact that the two unusually high values do not occur at similar points in the truss, suggests that strain gage results may be erroneous.

These secondary stresses are high at the ends of the members in the panel points. In designing the individual members as columns a reduction is made in the allowable unit stress depending on the "slenderness ratio". Most truss members are long and slender, so the unit stress is reduced to allow for bending at the center portion of the member. Since the secondary stresses are high at the supports, the reduction in allowable unit stress for bending allows for the added stress at the supports where the members are understressed. The combination of secondary stresses and column action tends to produce a balanced design in the member.

In the analysis of the glue areas at joint L_1 it was shown that the two forces acting on the glue area caused stresses in the starved glue joint above those adequate to good design. On some parts of a glue area these two forces add to each other and on others they counteract each other.

The twisting stress added to the axial stress at points O and P (Figure 10) and subtracted from the axial stress at points N and Q. This might explain the lacquer pattern in

Figure 20. The cracks are normally vertical under pure tension, but the effect of twisting at point O tended to bend these cracks perpendicular to the maximum stress. Since the starved glue area caused the effective glue to be concentrated over the first 1 3/4 inches of the joint, the strain in the gusset was greatest over this area. This high strain in the glue area accounts for the density of cracks in the lacquer over the starved area. From this analysis, it appears that brittle lacquer can be used to give a qualitative analysis of the strain in wood.

At the design load, no cracking appeared in the lacquer and it was not until the load was increased to 340 psi fluid pressure that cracks appeared and the joint failed. The stress on the glue area for the side that failed was 763 psi and for the intact side was 168 psi. At the instant the joint failed, the entire load and moment shifted to the area still working, raising that unit stress to 790 psi. Due to the failure, however, the deflections in the truss were relieved, which immediately caused a reduction in the applied load. This prevented the remaining glue area from failing. It indicates that glue is capable of withstanding an instantaneous overload.

The use of 3/4 inch material for gusset plates is not recommended as it is too stiff to conform to members of different thickness, as was the case in truss number one. If the material had been 3/8 inch or 5/8 inch thick the nails would have drawn the material to the thin members, thus reducing the amount of area not glued.

The Review of Literature indicates a wide variation in the shear stress for glue. A unit stress of 200 psi is considered practical for farm buildings. The unit stress in shear for wood is 95 psi, and it was found that the failure occurred in the wood fibers due to the lower value for wood. The computed unit stress on the glue line area at failure was 763 psi or eight times the recommended design value which was considered to be the unit shear for wood.

Using 95 psi for shear provides a factor of safety large enough so that only one-eighth of the glue area would have to be effective to support the design load. If plywood were used and designed with the value for unit shear in wood, it would be adequate for gusset plates.

Appendix E shows a tabulation of the calculated and measured deflections of all panel points. In case of the calculated deflections the effect of the entire chord L_0U_2 acting as a beam was not included. A 2" x 8" member on a 20' span will not support a very large load without excessive deflection. If this beam effect were significant the calculated deflections would have been higher than the measured deflections but the calculated values were not consistently higher or consistently lower than the measured values.

From these data it appears that the effect of the beam action of the top chord two panels in length did not introduce measurable effects so far as truss deflection is concerned.

Truss action, on the other hand, did produce a very definite effect upon the action of the top chord. With the middle support of the top chord removed and with a value of 1,600,000 psi for the modulus of elasticity of wood, the vertical deflection of the mid-point (U_1 on the truss) would be approximately 4" against an observed truss deflecion of 0.210".

CONCLUSIONS

1. The SR-4 strain gage is useful in determining quantitatively the strain in wood. Laboratory tests on a limited number of samples indicate that accuracies of less than 10 per cent are possible. Further tests are needed to determine the accuracy and other characteristics of the gage when used on wood which is non-homogeneous. The effect of differences in grain directly under the gage should be determined. In this investigation very small strains were measured so the variation in terms of per cent would be high although actual values would be relatively small (page 75).

   Brittle lacquer is useful in determining the distribution of stress. Usually the strain cracks appear perpendicular to the direction of stress caused by tension and parallel to the stress caused by compression. The creep in the lacquer is usually small however in this study it appeared that the creep in the lacquer was approximately the same as the strain in the wood. Instead of a gradual increase in the number of cracks they all appeared within a period of several seconds. Experience in application of the lacquer would make the results more consistent and easier to interpret. The type of lacquer could also be improved so it could be used on the job site with greater ease and accuracy (page 67).
2. The computed values for deflection were made with the

assumption that no rotation occurred at the panel points so that θ_A and θ_B equal zero. The computed values of strain check very well with the measured values which indicates that the assumption is logical and could be used in future calculations. The observed deformation of the member was the same as that predicted from computation (Appendix E).

When bending in the top chord members of this truss is calculated and combined with the axial stress to produce primary stress, the magnitude of the secondary stress is less than 17.6 per cent of the primary stress. If the allowable unit stress in members is decreased according to the standard column formula, the secondary stresses will be taken care of, as the decrease in allowable stress allows for bending at the center of the member. At the ends this allowance tends to offset the added stress due to secondary stresses causing bending.

In the design of this type of truss the secondary stresses could be neglected when using presently accepted standards as the factors of safety and design procedures make adequate allowances (page 77).

3. In the use of a design unit stress of 95 psi (recommended in reference 5 for shear in wood) for glue line shear, the factor of safety is from six to eight. In this truss design, therefore, both the glue and the wood in the plywood have a large factor of safety. For farm buildings

this factor of safety could be reduced to make the materials more efficient. A design stress of 200 psi would be adequate and would result in factors of safety approximately the same as those used in other designs (page 79).

4. The Review of Literature indicates a general agreement in the type of pressure on low pitched roofs; however, the value for pressure coefficients varies considerably. For this study the following coefficients were applied to the wind pressure on a flat vertical surface to obtain loads on the roof:

upper $\frac{1}{2}$ of leeward side of roof	-0.5
lower $\frac{1}{2}$ of leeward side of roof	-0.5
lower $\frac{1}{2}$ of windward side of roof	+0.3
upper $\frac{1}{2}$ of windward side of roof	-0.2 (page 5).

5. The use of hydraulic cylinders for loading a truss is probably the most nearly accurate and the most practical method available. Small or large loads can be applied quickly and with equal accuracy (page 49).

SUMMARY

The major objective of this investigation was to determine the importance of secondary stress in a truss with rigid joints, and what allowance should be made in the design of this type of truss.

A theoretical analysis was made of a type of truss which is common for light construction in the North Central Region. A check of two types of loading indicated that dead load plus snow load caused maximum stress in the members. For this reason a dead plus snow load was used in the experimental investigation. Stresses due to axial loads and bending were calculated.

Preliminary tests were run (1) to determine the practicability of using SR-4 electric resistance strain gages on wood, (2) to investigate the possibility of using brittle lacquer to locate points of maximum strain, (3) to find a practical method of applying load to a full scale truss, and (4) to determine the modulus of elasticity to be used in calculating the stresses from the measured strains.

A preliminary truss was used to check the apparatus and the use of brittle lacquer in locating areas of maximum strain. When a load of approximately two and one-half times the design load was applied, member L_1L_0 pulled loose at joint L_1. Investigation showed that imperfect construction caused the failure. The failure provided information for theoretically

determining the stress on the glue line area and it appeared that the unit stresses in that area were approximately eight times the allowable design value of 95 psi.

The final truss was placed in the testing frame and loaded to nearly two times the design load but not to the point of failure. The load was applied in increments of 133 pounds per loading point, which was 50 psi fluid pressure. This increment could be read easily on the gages so that the load could be kept more constant during the time the readings were taken.

SR-4 strain gages were used to measure unit strains on the four surfaces of the ends of each member. The measured axial strain was found by averaging these four values. The strain due to bending was computed as one-half the difference between the strain on the top surface and the bottom surface (when the truss is in a vertical position). The experimental values were compared with calculated values as shown in Table 11. Within the limits of the instruments and apparatus, the calculated and measured values give similar results.

SUGGESTIONS FOR FURTHER STUDY

1. Investigate the possibility of using other materials for gusset plates.
2. Conduct similar analyses on different types of trusses.
3. Conduct a more complete study of the use of strain gages for measuring stresses in wood.
4. Determine the action of the truss over a longer period of time.
5. Work out practical procedures for mass production of trusses.
6. Make a complete analysis of the stresses which occur in the gusset plates.

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High School: Volga PS, Volga, South Dakota
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Civic Activities:

College Lutheran Church-Member Council	1948-1950
Member of church choir	1951-1953
Member Lutheran Student Foundation	1948-1950
Secretary - 1949	
Member Lansing Orpheus Club	1947-1950
Member East Lansing PTA	
County CROP Committee	1950
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Military Service:

4 Months training Navy Diesel School, Cornell University
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APPENDICES

Appendix A.

Data recorded in preliminary tests to calibrate hydraulic cylinders. Loads exerted by the various cylinders at the pressure indicated. Pounds.

Fluid Pressure Psi	Cylinder Number 1	2	3	4	5	6	7	8	9	10	11	Max. diff. lbs.
50	117	108	109	105	112	108	108	109	109	110	104	13
100	254	249	248	242	252	248	252	254	248	250	242	12
150	395	389	388	384	394	394	393	395	[illegible]	391	380	15
200	526	524	526	525	531	529	531	533	527	529	516	17
250	670	659	663	661	672	669	668	672	665	668	652	19
300	804	794	799	796	808	806	804	811	803	806	789	19
350	939	928	939	935	948	946	943	947	940	946	928	23
400	1075	1064	1076	1071	1084	1085	1082	1086	1076	1083	1065	28
450	1223	1208	1218	1215	1231	1229	1226	1230	1223	1229	1210	25
400	1122	1131	1120	1120	1141	1128	1139	1138	1128	1132	1127	19
350	992	1004	987	1008	997	993	1001	999	997	991	1012	17
300	856	868	848	866	855	852	856	855	857	844	882	38
250	708	724	707	726	713	710	714	714	711	704	745	41
200	571	577	568	584	566	565	571	573	568	565	592	27
150	438	436	423	443	425	424	426	430	425	421	451	23
100	291	281	272	294	276	273	278	278	280	274	294	22
50	139	135	129	146	126	129	133	129	135	134	150	20

Appendix B

Code used for locating strain gages on the truss members.

Switch No. 1

Gage No.	Member	Location
A_1	L_0U_1	Top side
A_2		Top
A_3		Bottom side
A_4		Bottom
A_5	L_0L_1	Top side
A_6		Bottom
A_7		Bottom side
A_8		Top
A_9	L_0	Rosette 1
A_{10}		Rosette 3
A_{11}		Rosette 4
B_1	U_1L_0	Top side
B_2		Top
B_3		Bottom side
B_4		Bottom
B_5	U_1U_2	Top side
B_6		Top
B_7		Bottom side
B_8		Bottom
B_9	L_1	Rosette 3
B_{10}		Rosette 4
B_{11}		Rosette 1
C_1	L_1L_0	Top side
C_2		Bottom
C_3		Bottom side
C_4		Top
C_5	U_1L_1	Top side
C_6		Top
C_7		Bottom side
C_8		Bottom
C_9	L_1U_2	Top side
C_{10}		Top
C_{11}		Bottom side
D_1	L_1U_1	Top side
D_2		Top
D_3		Bottom side
D_4		Bottom
D_5	L_1L_2	Top side
D_6		Bottom
D_7		Bottom side
D_8		Top
D_9	L_1	Rosette 2
D_{10}	L_1U_2	Bottom
D_{11}	U_1	Parallel to U_1L_1
E_1	U_2U_1	Top side
E_2		Bottom
E_3		Bottom side
E_4		Top #1
E_5	U_2L_1	Top side
E_6		Bottom
E_7		Bottom side
E_8		Top
E_9	U_2U_1	Top #2
E_{10}	U_2	Gusset perpen. to L_1U_2
E_{11}		Gusset parallel L_1U_2

Legend

Top--Upper surface of member when truss is erect.
Bottom--Lower surface of member when truss is erect.
Top side--Upper surface when truss is lying on the ground.
Bottom side--Lower surface when truss is lying on ground.
Rosette--Indicates one strain gage on a rosette. For orientation see sketch above.

Appendix B (Continued)

Switch 2

Gage No.	Member	Location
A_1	L_3U_3	Top side
A_2		Top
A_3		Bottom side
A_4		Bottom
A_5	L_3L_2	Top side
A_6		Bottom
A_7		Bottom side
A_8		Top
A_9	L_3	Rosette 1
A_{10}		Rosette 4
A_{11}		Rosette 3
B_1	U_3L_3	Top side
B_2		Top
B_3		Bottom side
B_4		Bottom
B_5	U_3U_2	Top side
B_6		Top
B_7		Bottom side
B_8		Bottom
B_9	L_2	Rosette 4
B_{10}		Rosette 3
B_{11}		Rosette 2
C_1	L_2L_3	Top side
C_2		Bottom
C_3		Bottom side
C_4		Top
C_5	U_3L_2	Top side
C_6		Top
C_7		Bottom side
C_8		Bottom
C_9	L_2U_2	Top side
C_{10}		Top
C_{11}		Bottom side
D_1	L_2U_3	Top side
D_2		Top
D_3		Bottom side
D_4		Bottom
D_5	L_2L_1	Top side
D_6		Bottom
D_7		Bottom side
D_8		Top
D_9	L_2	Rosette 1
D_{10}	L_2U_2	Bottom
D_{11}	U_3	Parallel to U_3L_2
E_1	U_2U_3	Top side
E_2		Top
E_3		Bottom side
E_4		Bottom
E_5	U_2L_2	Top side
E_6		Bottom
E_7		Bottom side
E_8		Top
E_9	L_3	Rosette 2
E_{10}	U_2	Gusset perpend. to U_2L_2
E_{11}		Gusset parallel to U_2L_2

APPENDIX D

DATA SHEET FOR TRUSS LOADING TESTS

OBSERVER J. S. Boyd TRUSS NO. 2 DATE July 7 / August 19

AT TIME OF CONSTRUCTION: REL. HUMID. 58 % TEMP. 81 °F

AT TIME OF TESTING: REL. HUMID. 50 % 80 °F

MEMBER	MOISTURE CONTENT AT TIME OF CONST'N	MOISTURE CONTENT AT TIME OF TESTING	WT./FT. AT CONST'N	% moisture on July 11, 1953
L_0U_2	16.0	12.5	2.85	13.5
U_0L_3	16.0	12.0	2.85	13.5
U_1L_1	13.0	12.0	1.28	11.5
U_2L_1	11.5	12.0	1.25	11.5
U_2L_2	12.0	12.0	1.25	12.0
U_3L_2	14.0	12.0	1.28	12.0
L_0L_1	11.5	12.0	1.34	10.0
L_1L_2	13.0	12.0	1.21	12.0
L_2L_3	13.0	11.0	1.26	10.0
GUSSETS	9.0	11.5		

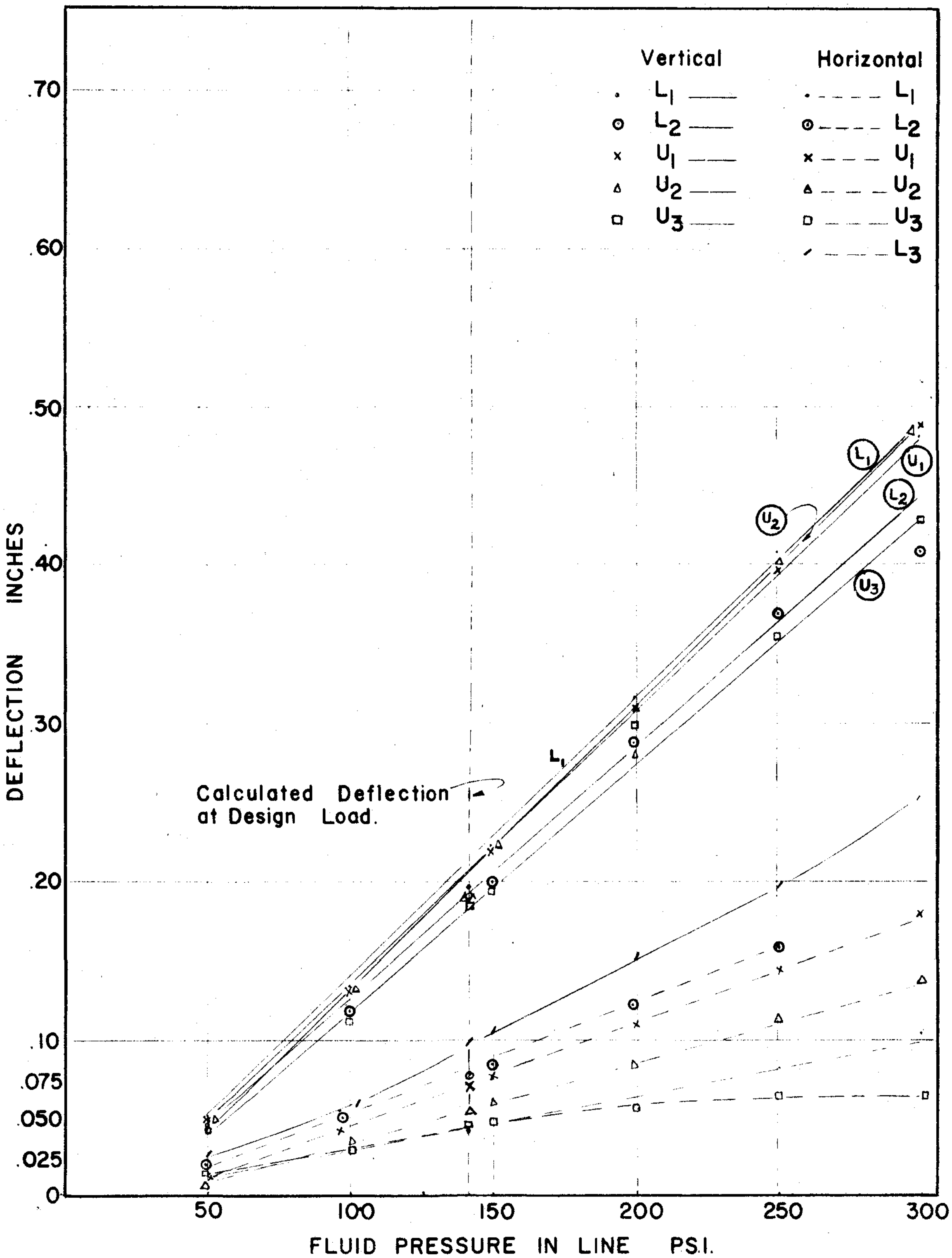
REMARKS ON August 18 T = 80°, Rel. Humid. = 40%

August 19 T = 80°, Rel. Humid. = 35%

August 21 T = 82°, Rel. Humid. = 38-40%

August 22 T = 60°, Rel. Humid. = 60% Start

T = 78°, Rel. Humid. = 40% Finish



APPENDIX E LOAD DEFLECTION CURVES FOR TRUSS JOINTS

Appendix F. Measured Deflection-Inches

Fluid Pres. in Cylinder	Run	L0 H	L0 V	L1 H	L1 V	L2 H	L2 V	L3 H	L3 V	U1 H	U1 V	U2 H	U2 V	U3 H	U3 V
50 psi	1	0	0	.014	.053	.020	.050	.029	0	.018	.050	.013	.052	.014	.045
	2	0	0	.010	.048	.016	.038	.021	0	.016	.046	.010	.043	.014	.038
	3	0	0	.013	.051	.020	.051	.025	0	.005	.047	.008	.048	.015	.042
	Av.	0	0	.012	.051	.019	.046	.025	0	.017	.048	.010	.048	.014	.043
100 psi	1	0	0	.028	.126	.032	.097	.053	0	.050	.129	.035	.122	.028	.098
	2	0	0	.029	.142	.052	.129	.068	0	.048	.137	.038	.140	.031	.121
	3	0	0	.030	.134	.053	.131	.067	0	.024	.130	.033	.134	.032	.118
	Av.	0	0	.029	.134	.046	.119	.063	0	.041	.132	.035	.132	.030	.112
200 psi	1	0	0	.060	.301	.118	.250	.126	0	.118	.302	.087	.185	.014	.243
	2	0	0	.066	.329	.123	.306	.163	0	.118	.318	.083	.333	.062	.296
	3	0	0	.065	.319	.124	.301	.161	0	.091	.306	.073	.322	.061	.291
	Av.	0	0	.063	.316	.122	.303	.150	0	.109	.309	.081	.280	.056	.277
300 psi	1	0	0	.097	.495	.223	.430	.235	0	.187	.481	.134	.467	.049	.423
	2	0	0	.105	.560	.228	.422	.269	0	.190	.498	.131	.509	.071	.441
	3	0	0	.107	.453	.201	.373	.259	0	.161	.481	.145	.490	.075	.418
	Av.	0	0	.103	.503	.217	.408	.254	0	.179	.487	.137	.489	.065	.427

Appendix G. Comparison of

Measured vs. Calculated

Deflections

Joint	Calculated H	Calculated V	Measured H	Measured V
L_0	0	0	0	0
L_1	.039	.194	.040	.215
L_2	.071	.196	.080	.190
L_3	.109	.000	.095	.000
U_1	.069	.186	.070	.210
U_2	.054	.186	.060	.210
U_3	.039	.184	.045	.180